THE EFFECTS OF TRUCK PLATOONS ON STEEL BRIDGE LOAD RATINGS

A Thesis

by

RITA TOHME

Submitted to the Office of Graduate and Professional Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Chair of Committee,	Matthew Yarnold
Committee Members,	Amir Behzadan
	Peter Keating
Head of Department,	Robin Autenrieth

May 2019

Major Subject: Civil Engineering

ABSTRACT

Platooning is the use of vehicle-to-vehicle communications and sensors, such as cameras and radars, to allow two or more trucks to drive as a single unit, and automatically accelerate and brake together, allowing them to travel at closer distances. With the world moving closer towards a more environmental-friendly approach to everyday decisions, it is not a surprise that the concept of truck platooning is gaining momentum, as it reduces CO2 emissions by lowering fuel consumption. However, studies need to be performed to confirm that bridges already in existence will be able to handle platoons, even though they were not designed for them, or, restrict platoon activity on the ones that show poor results.

The scope of this research is to study the effects of truck platooning on, steel girder bridges of single, two, and three-spans. Several truck-to-truck distances were selected and tested with a different number of trucks, per platoon, while varying spans lengths, or girder spacings. The AASHTO design and legal load ratings were calculated for each platoon case and were then used to examine the effects of truck platooning on bridge load ratings, and thus the adequacy of current bridges to carry platoons.

The overall findings of the study include the following observations. Bridges previously designed using Allowable Stress Design may be inadequate for truck platoons in the positive moment region for medium to long-spans. In addition, they may be inadequate in the negative moment region for short spans. Bridges previously designed

using Load Factor Design will likely be adequate for truck platoons in the positive moment region. However, issues may arise in the negative moment region for medium span lengths. Bridges previously designed using Load and Resistance Factor Design may be inadequate for truck platoons in the positive moment region for long-spans. However, there will likely not be any issues in the negative moment region. The number of trucks within a platoon and the spacing between trucks inside a platoon are the most influential parameters on their load ratings. The adequacy of existing bridges will hinge on the quantities of these variables.

DEDICATION

To my parents, Rima and Daoud, this would have not been possible without you.

Thank you for giving me the world.

ACKNOWLEDGEMENTS

I would like to thank my committee chair, Dr. Matthew Yarnold, not only for his guidance and support throughout the course of this research, but also for all his time and commitment to helping me produce a quality study.

I would like to also thank my committee members, Dr. Peter Keating, and Dr. Amir Behzadan, for their guidance and support throughout the course of this research.

Thanks also to my boyfriend, George, for his constant support and his patience.

Lastly, and mostly, thank you to my mother and father, Rima and Daoud, for their endless encouragement of all my endeavors.

CONTRIBUTORS AND FUNDING SOURCES

This work was supervised by a thesis committee consisting of Doctor Matthew Yarnold and Doctor Peter Keating of the Department of Civil Engineering and Doctor Amir Behzadan of Construction Science. All work for the thesis was completed independently by the student. This graduate study was supported by a fellowship from Texas A&M University. Also, Freese and Nichols, funded a great portion of the tuition associated with the degree obtained from this research.

TABLE OF CONTENTS

Page

LIST OF FIGURES

LIST OF TABLES

NOMENCLATURE

AASHTO	American Association of State Highway and Transportation Officials
ACC	Adaptive Cruise Control
AISC	American Institute of Steel Construction
ASR	Allowable Stress Rating
CACC	Cooperative Adaptive Cruise Control
CO2	Carbon Dioxide
LFR	Load Factor Rating
LRFD	Load and Resistance Factor Design
LRFR	Load and Resistance Factor Rating
MBE	Manual for Bridge evaluation
RF	Rating Factor

1. INTRODUCTION

1.1. Truck Platooning

Platooning is the use of vehicle-to-vehicle communications and sensors, such as cameras and radars, to allow two or more trucks to drive as a single unit. All the trucks within that unit automatically accelerate and brake together, allowing them to travel at closer distances. According to the Office of Energy Efficiency and Renewable Energy, the platooning technology detects and reacts to changes in the environment automatically. When the lead truck slows down, that speed is almost instantaneously implemented on the trucks in the rest of the fleet, traveling together as one unit. There are many variations, and types of platooning coming to light. They differ in some elements, such as the involvement of the drivers of the trailing trucks, the involvement of the driver of the leading truck, and the equipment used to communicate between the trucks. However, they all aim to achieve a main physical goal of allowing the trucks to be closer to each other. (“Platooning Trucks to Cut Cost and Improve Efficiency”, 2018).

1.2. Benefits of Truck Platooning

With the world moving closer towards a more environmental-friendly approach to everyday decisions, and an accepting attitude towards technological advancement, it is not a surprise that the concept of truck platooning is gaining momentum. Truck platooning reduces CO2 emissions by lowering fuel consumption. According to the European Automobile Manufacturers Association, CO2 levels are reduced by up to 16% from the trailing truck, and up to 8% from the leading truck. That lower fuel consumption is mainly due to the significant reduction of air-drag friction from the trucks driving closer to one another. That results in saving money and reducing pollution. Platooning also improves safety, since braking is automatic and thus immediate. A safe distance is usually preserved between two vehicles to account for the stopping distance needed by giving the driver enough time to perceive an incident, and enough time to react to it. That extra distance can be safely eliminated since perception and reaction are eliminated by having the two-trucks linked to one another. ("What is Truck Platooning?", 2017).

1.3. Truck Platooning Feasibility

When it comes to feasibility of truck platooning, the technology is present. There is a variety of platooning options with different technologies and methodologies available. Most new class 8 trucks are equipped with adaptive cruise control (ACC) that automatically adjusts the truck speed to maintain a safe driving distance to achieve the minimum headway clearance from the vehicle in front of it. "Safety equipment like automatic braking and lane keeping are options fleets can buy on their trucks, and they are being bought on a pretty high scale with no regulations requiring them. A lot of the technology that is required to platoon two-trucks is already on the truck. Now we just have to figure out how to handle the vehicle-to-vehicle communication." ("What is Truck Platooning?", 2017). Using commercially available ACC and collision avoidance systems, vehicle-to-vehicle communication can enable cooperative adaptive cruise control (CACC).

1.4. Truck Platooning Challenges

One of the challenges truck platooning is facing is government agencies acceptances. Government agencies were skeptical at first. However, some states have already legalized truck platooning, and others are closely behind. Figure 1 retrieved from the National Conference of State Legislatures, summarizes the status of all the states on truck platoon legal acceptance. More than half of the states have either enacted legislation, or an passed an executive order, relating to truck platooning. ("Self-Driving Vehicles Enacted Legislation ", 2018).

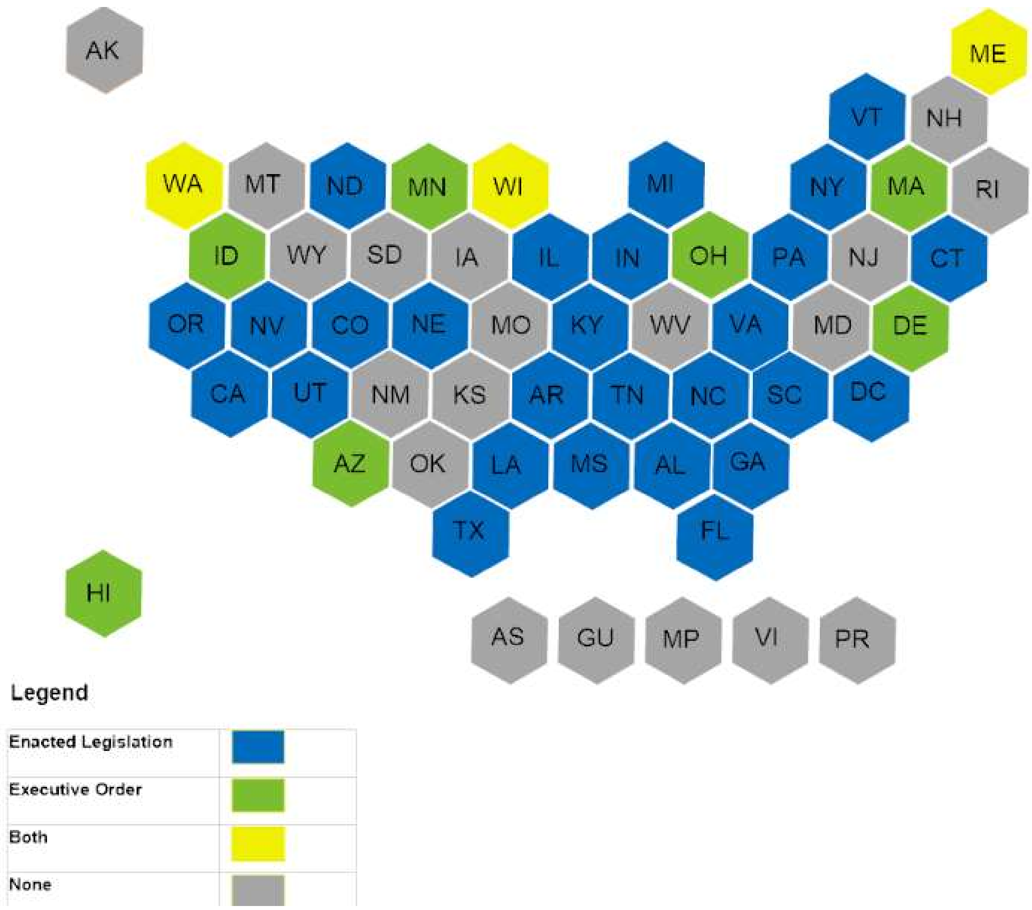


Figure 1: States with Autonomous Vehicles Enacted Legislation and Executive Orders

Afterall, the benefits of a greener environment due to the reduction of CO2, lower consumption of fuel, safer roads, and less congested roads, are all immediate benefits of truck platooning. Another potential challenge is public acceptance. Motor vehicle drivers are always expected to keep a safe driving distance. The public might be alarmed at seeing large trucks driving so closely together. That might lead them to be distracted, and even call the cops. Another challenge related to the public acceptance, is motor vehicle drivers disturbing the platoon. If a car merges between two-trucks, it disturbs the flow, and the trailing truck will have to increase its following distance until the car as left the platoon unit. However, acceptance will come with time and education.

1.5. Designing for Truck Platoons

When truck platooning becomes more eminent, design engineers will have to revisit some of their codes, particularly, bridge and pavement design codes. However, a more pressing matter is the existing pavements and bridges that were designed before the thought of platoons came out. Studies need to be performed to confirm that pavements and bridges already in existence will be able to handle platoons, even though they were not designed for them, or, restrict platoon activity on the ones that show poor results.

1.6. Load Rating

“The Load Rating is a measure of bridge live load capacity and has two commonly used categories: Inventory Rating, and Operating Rating”. Inventory rating includes loads in multiple lanes that can utilize the bridge for an indefinite period. The operating rating is the maximum live load that can be placed on the bridge in multiple lanes. It is expressed in terms of an equivalent HS-truck. However, allowing unlimited usage at the operating rating will reduce the life of the bridge. (“Bridge Inspection Manual: Load Ratings” (n.d.)).

1.7. Research Scope

The scope of this research is to study the effects of truck platooning on composite and non-composite, steel girder bridges of single and multiple spans. Several truck-to-truck distances (or gaps) were selected and tested with a different number of trucks, per platoon. The AASHTO (American Association of State and Highway Transportation Officials) design load ratings were calculated for each platoon case, as well as the legal load ratings for each bridge. The design and legal load ratings were then used to examine the effects of truck platooning on bridge load ratings, and thus the adequacy of current bridges to carry platoons.

2. LITERATURE REVIEW

To begin with, it is beneficial and vital to examine previous works and research performed that relates to this study. Works and information related to the current AASHTO live load model, using that live load model for bridge evaluation, or ratings, and recent research specifically on truck platoons was sought out. This section summarizes relevant findings as a starting point for this work.

2.1. AASHTO Live Loads

In designing bridges, the live loads currently considered are the AASHTO live loads. The American Association of State and Highway Transportation Officials (AASHTO) has a series of specifications for truck loadings. However, the history of evolution of these live loads is interesting and related to this study, as it was based on truck "trains". The following figures, borrowed from John M. Kulichiki's presentation at University of Buffalo, wil illustrate the evaluation of the AASHTO live loads. In 1923, the proposed live load model, shown in Figure 2, was "Shoemaker's Truck Train and Equivalent Load" and was based on five trucks in a train, with their axle to axle spacing being 30 ft. The equivalent load from that train was determined to be a 600 lb/ft lane load, with a 28,000 lb concentrated point load. In a conference in 1929, the equivalent lane load from the truck train was revised. The lane load was increased to 640 lb/ft, and the concentrated point load was assigned different values for moment and shear. In determining the maximum moment, a point load of 18,000 lb was used; and when determining the maximum shear, a 26,000 lb point load was used, shown in Figure 3.

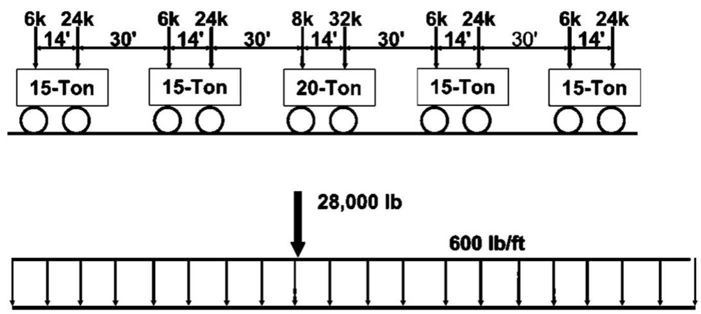


Figure 2: Shoemaker's Truck Train and Equivalent Load – 1923

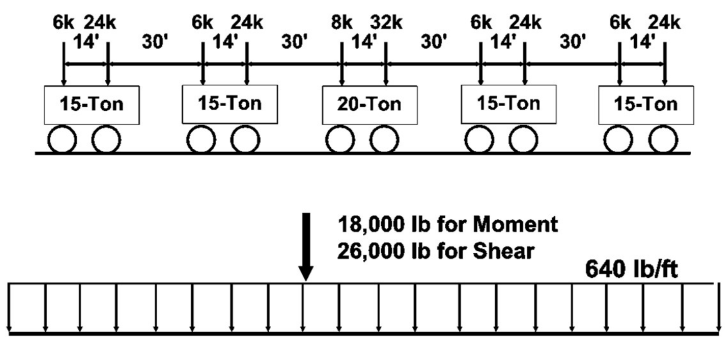


Figure 3: 1929 Conference Specification

Still, in 1941, the loads considered for the AASHTO Live loads evolved. The loads were reduced to examining a single notional truck, the HS20. The equivalent distributed load remained 640 lb/ft, but the point load was readjusted. For maximum moment effects, the point load was taken as 32,000 lb, and for maximum shear effects, the point load was taken as 40,000 lb. Figure 4 summarizes those configurations. Those loads are the loads documented in the AASHTO Standard Specifications.

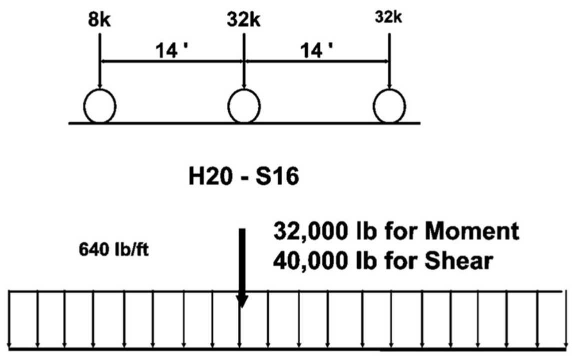


Figure 4: 1941, AASHTO HS20

Nonetheless, there was a lot of disagreement between design engineers on the validity of the HS20 configuration. Numerous studies were performed, and conferences were held. After careful and detailed studies, the AASHTO live loads were revised again. That final revision resulted in the HL-93 live load found today in the AASHTO LRFD Bridge Design Specifications. The HL-93 is a notional truck load, which uses the worse effect between a 2-axle tandem or a 3-axle truck, in addition to a 640 lb/ft lane load. Also, the HL-93 load includes the loaf of 90% of two-trucks spaced at 50 ft, or less, when calculating the negative moments. A summary of those configurations is shown in Figure 5, and Figure 6.

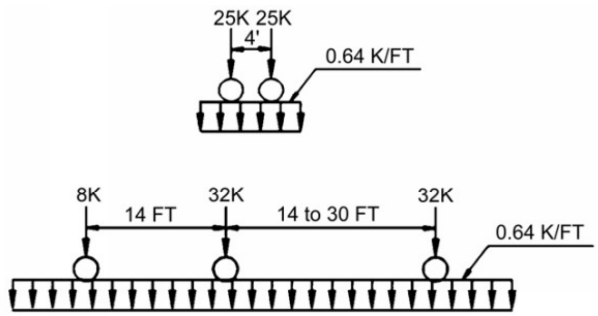


Figure 5: HL-93 Notional Load

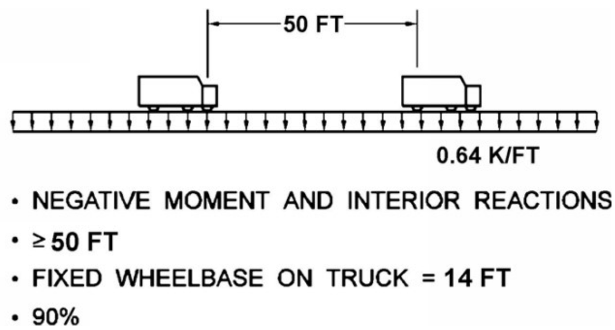


Figure 6: HL-93 Notional Load for Negative Moment

That represents a brief history of the evolution of the AASHTO live load, leading up to the HL-93. That history demonstrates that some platoons were considered in the development of the live load. However, those platoons, referred to as "trucks" did not consider the events of trucks driving at a close distance between each other due to advancement in technology. Those trains were considered at a reasonable and safe driving distances for trucks operated by human drives. Therefore, even though trains were considered, the load effect from those truck is not as big as the load effect that will be produced from the automation of truck platoons and reduction of the axle to axle spacing between trucks.

2.2. Load Ratings

The Manual for Bridge Evaluation, MBE, guides the calculation of load ratings for bridges, by providing a formula to calculate a rating factor, RF. This rating factor helps in the determination of the live load carrying capacity of bridges. The MBE includes three analytical load rating methods: Allowable Stress Rating (ASR), Load, Factor Rating (LFR), and Load and Resistance Factor Rating (LRFR), with the LRFR being the latest one to be presented.

Because of the different approved methods included in the MBE, and because the LRFR is new and did not exist when older bridges were designed, bridges standing today represent a mixture of bridges that were designed using different methods that are all approved under the national code.

In his study on the state of practice on the load rating of highway bridges in the United states, Lubin Gao conducted a survey to get a better understanding on the divide on the methods on load ratings that are predominantly used by different states. The results of a national survey, conducted in September of 2011, show that, for their future designs, 92% of the States use LRFR, where 40% use LRFR with AASHTO LRFD, and 52% have their own State-specific policies and procedures in place to implement the LRFR. (Gao, 2013)

Mertz also conducted a study to examine some differences between the Load and Resistance Factor Rating (LRFR) methodology and the Load Factor Rating (LFR) methodology. The study focuses on flexural-strength ratings of the interior girder of bridges. The study utilized the Virtis/Opis, the BRASS-GIRDER™, and the BRASS-

GIRDER(LRFD) software for the calculation of load ratings. From the findings, a ratio was obtained by dividing the LRFR by the LFR at inventory and operating level.

The bridges were grouped into different categories according to bridge type, and the means and standard deviations of the ratios (LRFD / LFR) in each category were obtained. As such, when the mean value is greater than one, the LRFR rating factor is greater than that of the LFR, and the opposite is true. Two of the categories of samples of examined bridges are relevant to this study: steel plate girder, and steel rolled beam.

For the steel plate girder category, the mean of the ratio of the LRFR to the LFR, considering the inventory case, is 1.19, and 0.93 considering the operating case. The standard deviation of that mean 0.21 in the inventory condition and 0.16 in the operating one. For the steel rolled beam category, the mean of the ratio of the LRFR to the LFR, considering the inventory case, is 1.05, and 0.8 considering the operating case. The standard deviation of that mean 0.42 in the inventory condition and 0.36 in the operating one. Therefore, the LRFRs are greater than the LFRs in the inventory conditions of both categories, but the LRFRs are smaller than the LFRs in the operating conditions of both categories. (Mertz, 2005)

In their study on the long-term effects of super heavy-weight vehicles on bridges, Wood, Akinci, and Liu, analyzed steel bridges using more than 50 sensors for 6 months. (Wood, 2007). Their analysis demonstrated that typical super load trucks up to a gross vehicle weight of 500 kips are not expected to cause any damage or impair the long-term performance of the investigated bridges. The strength limit states controlled the rating of steel bridges.

Still, Bourland, Chang, and Jao evaluated TxDOT criteria for the super heavy loads that trigger bridge analyses to determine whether the criteria adequately protect Texas bridges. A study of service life extension showed that operational stress level loads applied as little as 5 percent of time to a particular structure will have a significant effect on the lifetime of the structure. (Bourland, Chang, & Jao, 2011)

2.3. Truck Platooning

A study by Andrew Devault examined the effect of a two-truck platoon on selected bridges in the state of Florida. The clear bumper-to-bumper distance between the two-trucks studied was 30 ft, which is equivalent to a 40 ft distance between the truck axles. Two cases were analyzed, one where the weight of each of the two-trucks was 80,000 lb, and one where the weight of each truck was 88,000 lb. The 80,000 lb five axle combination truck used is a C5 truck, which is equivalent to a typical semi-tractor trailer. The 88,000 lb. truck results were obtained by multiplying the Rating Factors obtained from the 80,000 lb. platoon by the ratio of 80/88.

The findings of that study show that out of the 2,467 bridges that were analyzed, only 6 bridges were not able to accept the load of the 80,000 lb, two-truck platoon. However, out of the bridges studied, 22 bridges were not able to accept the load of the 88,000 lb two-truck platoon. Furthermore, when the bumper-to-bumper clear distance is increased to 60 ft, the 80,000 lb two-truck platoon could be safely driven on all the bridges included in the study. At the 60 ft clear bumper-to-bumper distance, 10 bridges out of the ones examined would not be able the handle the 88,000 lb. two-truck platoon. DeVault, A. (2017). Two-Truck Platooning. Load Effects of Two-Tuck Platoons on Interstate and Turnpike Bridges in Florida.

Additionally, Matthew Yarnold studied the live load effect of 20 ft to 40 ft distance spacings of two, three, four, and five truck platoons. Different span configurations were considered: single-span, two-span continuous where the two-spans were equal, and three-span continuous where the span length were 40% and 80% of the center span. All these configurations were checked using the LRFD AASHTO Bridge Design Specification, and the AASHTO Standard Specification of Highway Bridges. Similar to the study by Andrew Devault, the C5 truck was chosen for the study. A 40 ft axle-to-axle distance, which is equivalent to a 30 ft clear bumper-to-bumper spacing was examined. Similarly, a 20 ft axle-to-axle distance, which is equivalent to a 10 ft clear bumper-to-bumper spacing was also studied. Increments in between the 20 ft and 40 ft platoons were also considered.

At the 30 ft clear bumper-to-bumper spacing, most configurations checked under the AASHTO Standard Specification showed to be inadequate. The results increased in deviation from the Standard Specification with an increase in span length. Single-span bridges checked under the LRFD AASHTO proved to be more conservative. Two-span, and three-span bridges checked under that configuration had a similar trend except for three and four-truck platoons at longer spans.

At the 10 ft clear bumper-to-bumper spacing, the single-span results start showing a gap between the LRFD specs and the findings, notably at spans larger than 160 ft. At the single-span, the deviation becomes larger for the Standard Specifications case than at the 30 ft clear bumper-to-bumper distance. The two-span and three-span configurations also show reasons for concern in both the LRFD and the Standard Specification. Yarnold, (2018).

Both, the studies by Andrew Devault and by Matthew Yarnold show reasons for concern, as many of the existing bridges were not designed to handle the platoon loads, they will soon be exposed to, and the probability of failure is unknown. Short to moderate length of single-spans designed under the LRFD method have a better chance of being adequate to handling platoons. Beyond that, the clear bumper-to-bumper distance between trucks in a platoon is a major factor affecting the bridge's probability of failure under loading.

3. METHODOLOGY

This section explains the methodology used to perform the road rating evaluations. The load rating equation is presented, and its components explained. Also, relevant information on the determination of moment and shear on 2D line girder analysis is addressed. The standard procedures adopted are mentioned for the reader's references. Also, the excel program developed to perform the load rating evaluations is discussed. Moreover, the different types of load ratings, are discussed. The platoons chosen to be studied are defined, and the bridges chosen for the platoons to be evacuated on are also described.

In addition, Federal law states that two or more consecutive axles may not exceed the weight computed by the Bridge Formula even though single axles, tandem axles, and gross vehicle weights are within legal limits. Therefore, the Bridge Formula is examined to show if it would be enough to restrict platoons from operating on bridges.

3.1. Load Rating Equation

The load rating equation yields a rating factor through manipulation of the capacity and demand of a given member. The demand is represented as axial, flexure, or shear. Each member within a bridge has its own rating factor, and the lowest value was used as the prevailing rating factor. The bridge load rating factor is the value obtained by using the load rating equation. A load rating of 1.0 exactly meets the code provisions, while a load rating factor greater than 1.0 conservative. A load rating is the minimum load rating factor for a bridge multiplied by the weight of the truck used to obtain that load rating factor.

The equation below is used to determine the load rating of each component of the bridges to be examined, under the LRFR methodology:

$$RF = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \qquad \textit{Equation 1}$$

For the strength Limit States:

$C = \varphi_c \varphi_s \varphi R_n$

Where the following lower limit shall apply:

$\varphi_c \varphi_s \geq 0.85$

For the service limit states:

$C = f_R$

Where:

RF = Rating Factor

C= Capacity

f_R = Allowable stress specified in the LRFD or Standard Specifications code

R_n = Nominal member resistance

DC = Dead load effect due to structural components and attachments

DW = Dead load effect due to wearing surface and utilities

P = Permanent loads other than dead loads

LL = Live load effect

IM = Dynamic load allowance

γ_{DC} = LRFD or Standard Specifications load factor for structural components

γ_{DW} = LRFD or Standard Specifications load factor for wearing

surfaces and utilities

γ_P = LRFD or Standard Specifications load factor for permanent loads

other than dead loads

γ_{LL} = Evaluation live load factor

φ_c = Condition factor

φ_s = System factor

φ = LRFD or Standard Specifications resistance factor

As for the ASR and the LFR methods, the RF equation is presented below:

$$RF = \frac{C - A_1 * D}{A_2 * L(1 + I)} \qquad \textit{Equation 2}$$

Where:

RF = the rating factor for the live load carrying capacity

C = the capacity of the member

D = the dead load effect of the member

L = the live load effect of the member

A_1 = factor for load loads

A_2 = factor for live loads

An excel program was developed for this study. This program calculates the rating factors for a bridge by calculating the capacity and demand of its elements based on input variables. It handles up to three-spans and 20 axle loads. It is developed for rolled steel shapes and built up girders, of composite and non-composite bridges.

The following sections clarify how the various components of the load rating formula are obtained under the different design methodologies of Load and Resistance Factor Rating (LRFR), Allowable Stress Rating (ASR), and Load Factor Maximum Rating (LFR).

3.2. Shear and Moment Determination

The resulting moment and shear from the bridge loading is needed for the determination of the rating factor.

The truck axle loads were moved by 1/100th of the total span length, and the reactions, moment, and shear were calculated at each of those truck locations. Also, each span was divided into 100 segments, where the moment and shear were being calculated at the nodes of those locations. Engineering judgement was exercised to determine that these divisions were enough to provide accurate results since conventional design software utilizes less discretization.

The computations relied on the concept of superposition of loads. Specifically, the distributed load was analyzed separately from the distributed axle loads. Reactions, on each pier, were determined separately for the uniformly distributed load and the truck axles, and the summation of those reactions was the final result of reactions on each of the piers.

Moreover, the theorem of three moments, Clapeyron's theorem, is used to calculate the reactions of the piers of the multiple span bridges. (Muthu, Ibrahim, Janardhana, & Vijayanand, 2017). The simplified version of Clapeyron's equation considers a uniform beam section, where the stiffness is constant, and assumes that there is no settlement of the supports. Under these conditions, the simplified equation is shown below:

$$M_A l_1 + 2M_B(l_{1+}l_2) + M_C l_2 = -6(\frac{A_1 x_1}{l_1} + \frac{A_2 x_2}{l_2}) \qquad \textit{Equation 3}$$

In the equation above, consider a two-span bridge, supported by three piers. The lengths of the two-spans are l_1, and l_2 respectively, from left to right. The supporting piers are pier A, pier B, and pier C, respectively, from left to right. The two-span continuous configuration is separated to two single-spans. The moment diagrams of each of those two-spans in determined. The area under the curve of the moment diagram of the first (left-most) single-span is A_1, and the distance from pier A to the center of the area under the curve is x_1. Similarly, the area under the curve of the moment diagram of the second (right-most) single-span is A_2, and the distance from pier C to the center of the area under the curve is x_2, illustrated in Figure 7 (Muthu, Ibrahim , Janardhana , & Vijayanand, 2017).

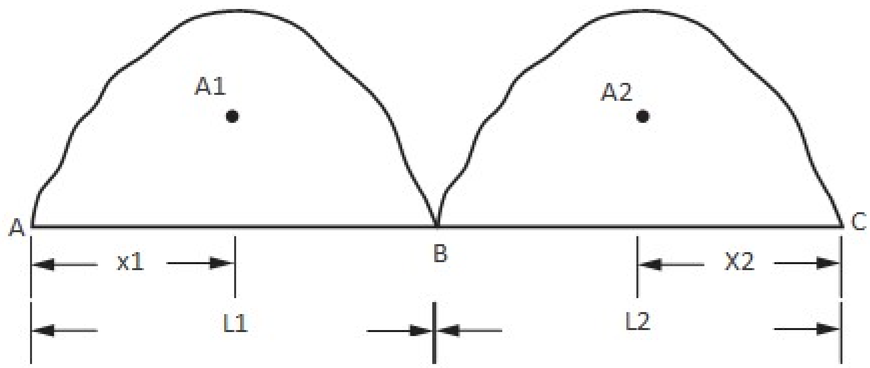


Figure 7: Two-Span Analysis Variables

$M_A, M_B,$ and M_C are the resulting moments of the two-span configuration. The summation of the moments on the exterior piers is 0. As such, $M_A = M_c = 0.$ The equation then simplifies to only one unknown, M_B. With the determination of M_B, the reaction of the pier B is determined from the two single-span configurations, and then added to obtain the complete reaction at pier B. Moment at pier A is taken as 0, and the reaction at pier C is calculated. Similarly, moment at pier C is taken as 0, and the reaction at pier A is calculated. The reactions are then checked by summing all the forces in the gravity direction, and checking that equilibrium is met.

For a three-span continuous configuration, a matrix needs to be solved. Consider a similar configuration to that of the two-span shown, with an additional pier D, and an additional span, l_3. This configuration is divided into two, two-span configurations, with the middle span being in both. The first two-span is made up of l_1, l_2. The second two-span is made up of l_2, l_3. $M_A = M_D = 0$. The two unknows that should be first evaluated are M_B and M_C, which are both part of the two-span configurations. As such, two equations with two unknows are formed. Solving the two equations will result in the moments at piers B and C to be known. Like the two-span configuration, from here the reactions at all four piers are determined.

With the reactions at the piers known, the moment and shear diagrams can be developed from a fixed location of the truck. Then, the truck is moved 1/100$^{\text{th}}$ of the total length of the spans, and this calculation is re-run, until the truck has assumed all the 100 positions that calculations are performed at.

At each of the 300 points that the moment and shear is calculated (100 per span), the maximum and minimum values for moment and shear are taken out of the 100 positions of the trucks. These maximum and minimum values represent the moment envelope and the shear envelope. From the envelopes, the maximum and minimum moment and shear can be extracted for internal and external bridge girders.

One of the main challenges of obtaining the moment and shear diagrams, is that a two-span and three-span bridge, is statically indeterminant. Therefore, summing moments at the supports to obtain the reactions to then develop the shear and moment diagrams is no longer enough. Clapeyron's theorem provides the additional equations required to solve the statically indeterminant problem.

Quickbridge is a free excel sheet, created by N. Turkkan, that provides the moment and shear envelops for multi-span bridges, under a uniformly distributed load and isolated axle loads. It was used as a reference and provides a medium for comparing and checking results of moment and shear values obtained from the excel sheet of this study. Testing the calculations revealed that the results match with those of Quickbride, with a 2% error. This error is acceptable for the purpose it is being used. Hand calculations were used to verify accuracy as well. Appendix A shows an example of a hand calculation to verify the results from the excel sheet. A third method to check these values is using a Finite Element software, STAAD, to compare results. When using the hand calculation and STAAD to check the accuracy. Appendix B shows a STTAD output result that matches the hand calculation presented in Appendix A. The main difference between those two checks and Quickbridge, is that a single position in time was used when using STAAD and hand calculations. However, Quickbridge results were compared to an envelope which is not a single position in time, but the maximum of all those positions. What was concluded, is that the 2% difference between the developed excel sheet and Quickbridge, lies in the specific discreet positions that are being used. Knowing that the difference is just a difference in the positions where the calculations are being performed, it is acceptable to conclude that the accuracy of the sheet used in this study has been verified, and the difference in results is accounted for, justified, and accepted. For further studies, if the accuracy of the sheet is needed to be improved, the calculations will need to be performed on more divisions that the current 100-secgement division being used. The way the sheet is set up makes it easy for this adjustment to be made in the future, if needed.

3.3. Standard Procedures Adopted

The procedures of calculating the bridge load ratings in this study, are adopted from the Manual for Bridge Evaluation (MBE), Second Edition. Specifically, these procedures are discussed in Section 6 of the MBE. The MBE was adopted by the American Association of State Highway and Transportation Officials (AASHTO) in 2005, and it provides multiple load rating methods. It represents a national standard in the United States. Part A of Section 6 of the MBE provides the criteria and procedures for the Load and Resistance Factor Rating (LRFR). Part B, on the other hand, incorporates provisions specific to the Allowable Stress Rating (ASR) and Load Factor Rating (LFR) methods for evaluation. Naturally, AASHTO LRFD Bridge Design Specifications, and the AASHTO Standard Specifications for Highway Bridges were used in this study to compliment the information provided by the MBE.

3.4. Excel Program

This section serves as a general guide to the procedures and methods used in the excel program for the load rating calculations. The spreadsheet includes the sections from the database of the American Institution for Steel Construction (AISC) manual and their section properties and develop the moment and shear diagrams of various dead and live load configurations that are used for the demand portion of the load rating. The spreadsheet follows the design standards of AASHTO Manual for Bridge Evaluation - 2018, AASHTO LRFD Bridge Design Specifications - 8th Edition, AASHTO Standard Specifications for

Highway Bridges - 17th Edition, and Texas Department of Transportation Bridge Design Manual - LRFD, to calculate the load rating of a bridge based on required user inputs.

The program was checked and authenticated by comparing results obtained from it to examples A1, and A4 from the MBE appendix. Also, results from the excel program were compared to the LRFD Design Example for Steel Girder Superstructure Bridge released by the Federal Highway Association.

The excel program was duplicated to be utilized for each of the configurations studied. Over 200 excel books were linked to each other, with one book being the main input book, and one book being the main output book. That means to run a simulation, one book had to be opened and have input altered, and then, all the results and graphs for all configurations were summed in one other excel book. However, all the other books had to be opened and saved for the results to be updated. Therefore, a VBA code was developed to open all the excel files and save them, one by one, in a correct chronological order.

The scope of the program is as follows: It calculates the load rating of steel bridges that are W-AISC shapes or built up girders. It adjusts for composite and non-composite sections. The load rating is checked using the LRFR, ASR, or LFR methodologies. The geometry of the bridge is limited to straight, not curved, bridges with a skew angle, and equal spacing of beams, up to 3 continuous spans, with up to 20 axle loads. Additional parameter ranges are ones imposed by LRFD Bridge Design Specifications and are as follows:

$$3.5 \leq S \leq 16$$

$$4.5 \leq t_s \leq 12$$

$$20 \leq L \leq 240$$

$$N_b \geq 4$$

$$10{,}000 \leq K_g \leq 7{,}000{,}000$$

$$-1.0 \leq d_e \leq 5.5$$

$$\theta \leq 60$$

Where:

d_e = horizontal distance from the centerline of the exterior web of the exterior beam at deck level to the interior edge of curb or traffic barrier (ft)

K_g = longitudinal stiffness parameter (in^4)

L = span of beam (ft)

N_b = number of beams, stringers, or girders

S = spacing of beams or webs (ft)

t_s = depth of concrete slab (in)

θ = skew angle (degrees)

3.5. Load Rating Calculation

3.5.1. Section Properties

As previously mentioned, the program was developed to handle W-shape and built-up girders. Section properties are extracted from AISC for W-shapes and are calculated for built up girders. Additionally, section properties for the composite and non-composite sections are calculated. If the bridge has a composite section, then short-term and long-term composite section properties are also evaluated. Under the composite design, two parameters are calculated: the effective flange width and the modular ratio.

3.5.1.1. Effective Flange Width

The total effective flange width is theoretically defined as the width of the slab that has a constant stress distribution equal to the maximum value of the actual stress distribution. The effective flange width is calculated differently under the AASHTO LRFD and the AASHTO Standard Specification. It is also different for interior and exterior girders. Per AASHTO LRFD Design 4.6.2.6.1, the effective flange width for an interior stringer or girder is the minimum of the equations below, where the variables are as defined on the previous page, except that the span length of the beam is converted from feet to inches, and:

t_w = thickness of the web (in)

$b_{f,top}$ = width of the top flange (in)

b_e = effective flange width (in)

The effective width of an interior girder or stringer under the LRFD method is:

$$b_{e,interior,LRFD} = \min(i, ii, iii) \quad \textit{Equation 4}$$

i. $\frac{1}{4} * L$

ii. $12 * t_s + \max\left(t_w \frac{1}{2} * b_{f,top}\right)$

iii. S

As for the effective flange width of the exterior stringer or girder, it is calculated as half of the effective flange length of the interior girder added to the minimum of the equations below, where the variables are as defined for the interior stringer or girder.

The effective width of an exterior girder or stringer under the LRFD method is:

$$b_{e,exterior,LRFD} = 0.5 * b_{e,interior,LRFD} + \min(iv, v, vi) \quad \textit{Equation 5}$$

iv. $\frac{1}{8} * L$

v. $6 * t_s + \max\left(\frac{1}{2} * t_w, \frac{1}{4} * b_{f,top}\right)$

vi. Overhang

Per AASHTO Standard Specifications 10.38.3, the effective flange width for an interior stringer or girder is the minimum of the equations below, with the variables and their units as defined for the effective width under the LRFD method. The effective width of an interior girder or stringer under the Standard Specifications method is:

$$b_{e,interior,Std} = \min(vii, viii, iv) \qquad \textit{Equation 6}$$

vii. $\frac{1}{4} * L$

viii. $12 * t_s$

ix. S

As for the effective flange width of the exterior stringer or girder, it is calculated as the minimum of the equations below, where the variables are as defined for the interior stringer or girder. The effective width of an exterior girder or stringer under the Standard Specifications method is:

$$b_{e,exterior,Std} = \min(x, xi, xii) \qquad \textit{Equation 7}$$

x. $\frac{1}{12} * L$

xi. $6 * t_s$

xii. $\frac{1}{2} * S$

3.5.1.2. Modular Ratio

The modular ratio, n, is typically defined as the ratio between the modulus of elasticity of steel and modulus of elasticity of concrete However, it is defined with a slight variation under the AASHTO Standard Specifications. This section property that is defined differently between the AASHTO LRFD and Standard. According to section C6.10.1.1.1b of the LRFD code, Table 1 summarizes the values of n, where f'_c, is the specified minimum 28-day compressive strength of the concrete is kilo pounds per square inches.

Table 1: Modular Ratio per AASHTO LRFD

$2.4 \leq f'_c < 2.9$	$n = 10$
$2.9 \leq f'_c < 3.6$	$n = 9$
$3.6 \leq f'_c < 4.6$	$n = 8$
$4.6 \leq f'_c < 6.0$	$n = 7$
$6.0 \leq f'_c$	$n = 6$

However, the modular ratio used for ASR and LFR evaluation, is presented in the MBE, section 6.B.5.2.4.1.1, as shown in Table 2.

Table 2: Modular Ratio per MBE

f'_c (psi)	n
2,000-2,400	15
2,500-2,900	12
3,000-3,900	10
4,000-4,900	8
5,000 or more	6

3.5.2. Dead Load Analysis

AASHTO divides its dead load classification into two major categories: DC and DW. Where DC is the dead load of structural components and nonstructural attachments, and DW is the dead load of wearing surfaces and utilities.

Furthermore, DC is divided into DC1 and DC2 under composite configurations. DC1 would include loads that will act as non-composite dead loads. That includes, but is not limited to, the weight of the deck, stringers, cover plates, diaphragms, haunch, stay-in-place forms, and stiffeners. DC2 includes composite dead loads such as the curb, parapet, and railing. Under non-composite construction, DC2 loads would be zero, and all the dead loads of structural components and nonstructural attachments would be included under DC1. DW includes the weight of the existing or future wearing surface.

3.5.3. Live Load Analysis

3.5.3.1. Live Loads

Under the LRFR method, the design live load is represented by an HL-93 notional truck. Appendix 6CA of the MBE includes a schematic of the HL-93 configuration, included in Figure 8. As the figure illustrates, the HL-93 notional truck is a 0.64 kilo pounds per linear foot design lane load, with either a 72 kilo-pounds design truck or a 50 kilo pounds design tandem, whichever produces the worse response. In addition, when evaluating the worse response for negative bending, 90% of the weight of two-trucks at a distance of 50 ft or more, is evaluated with the design load.

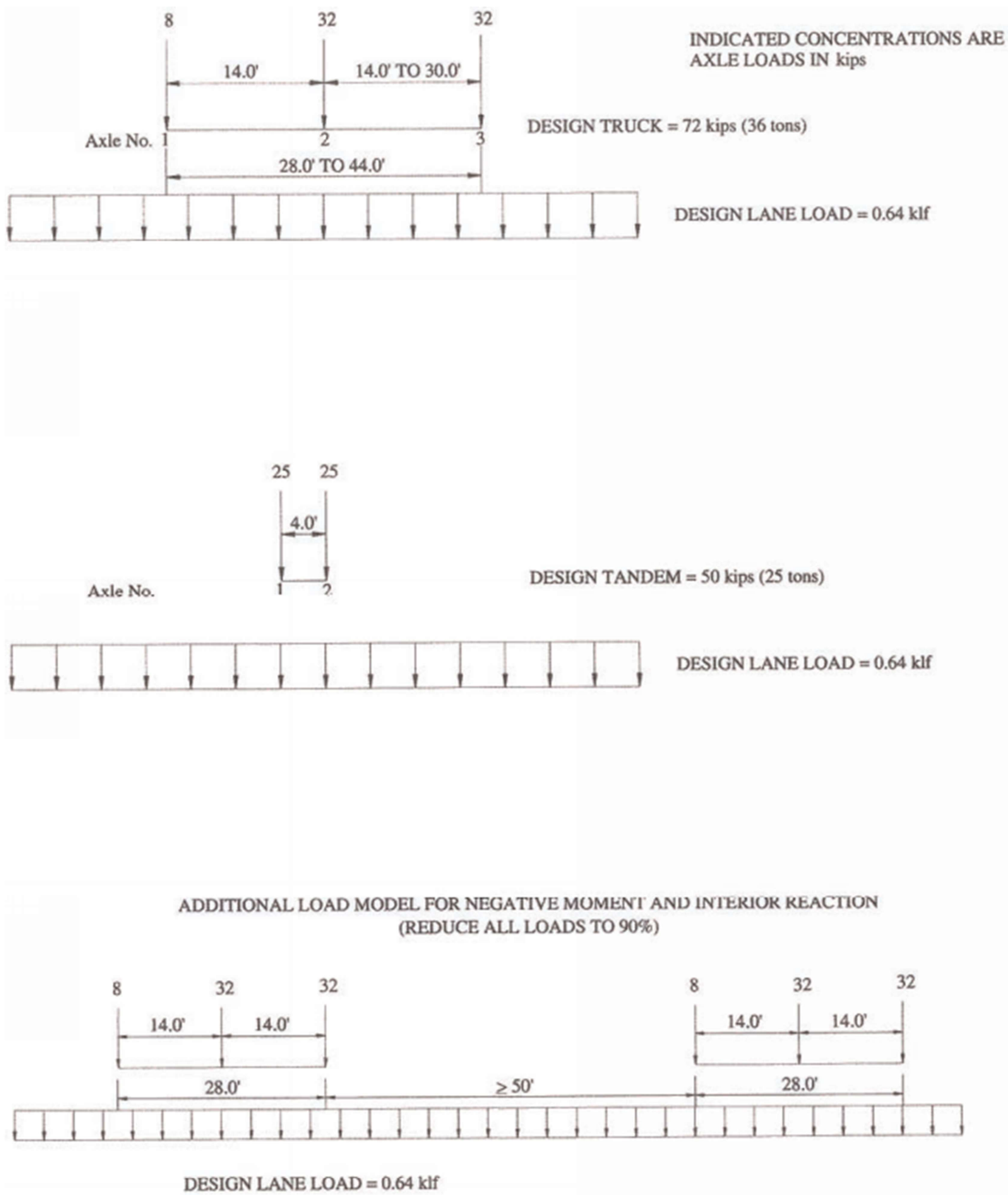


Figure 8: HL-93

Under the ASR method, the design live load is computed with an HS-20 notional truck. That configuration consists of taking the more conservative of two loadings. The first loading is a 72-kilo pound truck. The second configuration is a 0.64 pound per foot lane load, with a concentrated load placed at the location that causes the worst effect. When calculating the moment reaction, that point load is taken as 18 kilo pounds, and when calculating the shear reaction, that point load is taken as 26 kilo pounds. In addition, for the determination of the maximum negative moment in continuous spans, an additional second equal concentrated load should be placed in one other span, in such a way to create a maximum negative moment effect. AASHTO Standard Specifications Figure 3.7.7.A and Figure 3.7.6B include schematics of the HS-20 configuration. Figure 9,and Figure 10 illustrate the HS-20 notional truck.

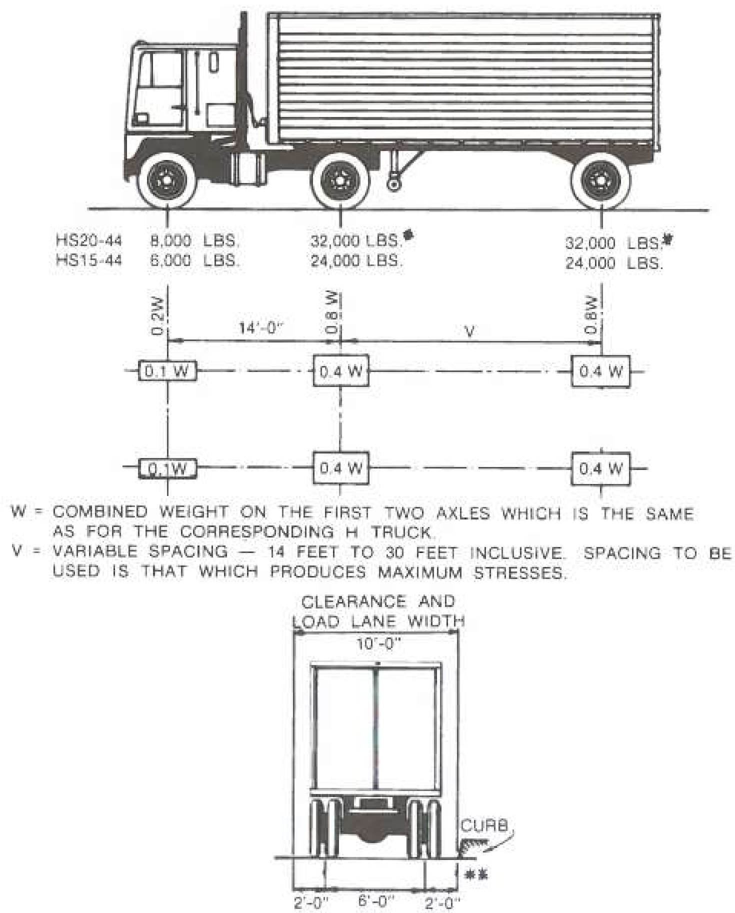


Figure 9: HS-20 Truck, AASHTO Standard Specifications 3.7.7A

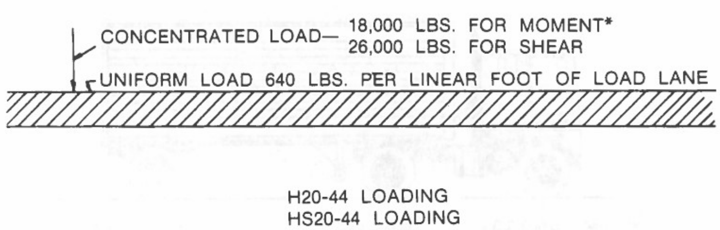


Figure 10: HS-20 Lane Load, AASHTO Standard Specifications 3.7.6B

3.5.3.2. Dynamic Load Allowance or Impact Fraction

To begin with, the Dynamic Load Allowance (AASHTO LRFD), or Impact Fraction (AASHTO Standard Specifications) is discussed. The dynamic load allowance (IM), as defined in AASHTO LRFD, is "An increase in the applied static force effects to account for the dynamic interaction between the bridge and moving vehicles." Per AASHTO LRFD 3.6.2.1, the dynamic load allowance is to be taken as 75% for deck joints in all limit states. For all other components, IM is taken as 15% for fatigue and fracture limit states, and as 33% for all other limit states.

The Impact Fraction, as given in AASHTO Standard Specifications Section 3.8.2 is defined as follows:

$$I = \frac{50}{L + 125} \leq 30\% \qquad \textit{Equation 8}$$

Where:

I = impact fraction

L= length in feet of the portion of the span that is loaded to produce the maximum stress in the member

3.5.3.3. Live Load Distribution Factors

A distribution factor estimates the percentage of the calculated load that is transferred to a single interior or exterior girder. In AASHTO LRFD, four distribution factors exist: moment interior, moment exterior, shear interior, and shear exterior.

AASHTO LRFD Table 4.6.2.2.2b-1 presents two equations to calculate the live load distribution factor of the moment in interior beams of steel bridges on concrete decks.

For one design lane loaded:

$$g_{M,I} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12 * L * t_s^3}\right)^{0.1} \qquad \textit{Equation 9}$$

For two or more design lanes loaded:

$$g_{M,I} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12 * L * t_s^3}\right)^{0.1} \qquad \textit{Equation 10}$$

Where:

$$K_g = n(I + A * e_g^2) \qquad \textit{Equation 11}$$

$$n = \frac{E_B}{E_D} \qquad \textit{Equation 12}$$

Where:

A = area of the stringer or beam (in^4)

e_g = distance between the centers of gravity of the beam and deck (in)

E_B = modulus of elasticity of the beam material (ksi)

E_D = modulus of elasticity of the deck material (ksi)

$g_{M,I}$ = moment distribution factor for interior girders (unitless)

$I =$ moment of inertia (in^4)

AASHTO LRFD Table 4.6.2.2.2d-1 presents an equation to calculate the live load distribution factor of the moment in exterior beams of steel bridges on concrete decks.

For one design lane loaded: Use the lever rule.

AASHTO LRFD defined the lever rule as "the statistical summation of moments about one point to calculate the reaction at a second point." When applying the lever rule, moments are summed about one support to find the reaction at another support by assuming that the supported component is hinged at interior supports. Figure 11, from AASHTO LRFD, shows how the notional load should be taken fir a three-girder bridge. Moments are taken about the notional hinge in the deck to find the reaction of the exterior girder.

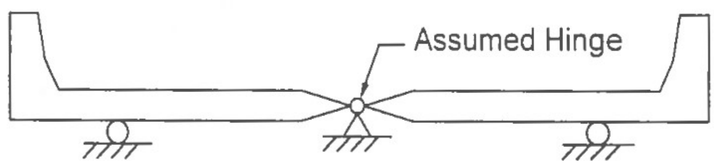


Figure 11: Notional Model for Applying Lever Rule to Three-Girder Bridges

For two or more design lanes loaded:

$$g_{M,E} = e * g_{M,I}$$ *Equation 13*

Where:

Equation 14

$$e = 0.77 + \frac{d_e}{9.1}$$

e = correction factor for distribution

$g_{M,E}$ = moment distribution factor for exterior girders (unitless)

The distance $d_e$shall be taken as positive id the exterior web inboard of the interior face of the traffic railing and negative if it is outboard of the curb or traffic barrier.

AASHTO LRFD Table 4.6.2.2.31-1 presents two equation to calculate the live load distribution factor of the shear in interior beams of steel bridges on concrete decks.

For one design lane loaded:

$$g_{V,I} = 0.36 + S/25 \qquad \textit{Equation 15}$$

For two or more design lanes loaded:

$$g_{V,I} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^2 \qquad \textit{Equation 16}$$

$g_{V,I}$ = shear distribution factor for interior girders (unitless)

AASHTO LRFD Table 4.6.2.2.31-1 presents two equation to calculate the live load distribution factor of the shear in exterior beams of steel bridges on concrete decks.

For one design lane loaded: Use the lever rule.

For two or more design lanes loaded:

$$g_{V,E} = e * g_{V,I} \qquad \textit{Equation 17}$$

Where:

$$e = 0.6 + \frac{d_e}{10} \qquad \textit{Equation 18}$$

$g_{V,E}$ = shear distribution factor for exterior girders (unitless)

The AASHTO Standard Specification provides an equation to calculate the moment distribution factor for interior stringers in Table 3.23.1. The three other distributions factors are calculated by the lever rule.

For one design lane loaded:

$$g_{M,I} = \frac{S}{4.5} \quad , \quad if\ t_s = 4\ in$$ *Equation 20*

$$g_{M,I} = \frac{S}{5.25} \quad , \quad if\ t_s \geq 6\ in$$ *Equation 19*

For two or more design lanes loaded:

$$g_{M,E} = \frac{S}{4} \quad , \quad if\ t_s = 4\ in$$ *Equation 22*

$$g_{M,E} = \frac{S}{5} \quad , \quad if\ t_s \geq 6\ in$$ *Equation 21*

3.5.3.4. Multiple Presence Factor or Reduction in Load Intensity

Multiple presence values account for the fact that the probability of events where ore lanes are loaded decreases as the number of lanes loaded increases. The live load distribution values that are calculated using equations that AASHTO LRFD and AASHTO Standard Specifications present are already adjusted for the multiple presence factor. However, when using the lever rule to obtain distribution factors, the multiple presence factors should be used for adjustment. AASHTO LRFD presents multiple presence factor values in Table 3.6.1.1.2-1, shown in Table 3.

Table 3: AASHTO LRFD Multiple Presence Factors

Number of Loaded Lanes	Multiple Presence Factors, m
1	1.20
2	1.00
3	0.85
>3	0.65

AASHTO Standard Specifications presents a similar concept but names it: Reduction in Load Intensity. The Standard Specification states, "Where maximum stresses are produced in any member by loading a number of traffic lanes simultaneously, percentages of live loads may be used in view of the improbability of coincident maximum loading." These reductions in load intensity are applicable when the lever rule is used and excludes the distribution factors obtained by using the formula from AASHTO Standard Specifications Table 2.23.1, presented in Table 4.

Table 4: AASHTO Standard Specifications Reduction in Load Intensity

Number of Lanes Loaded	Percent
One or two lanes	100
Three lanes	90
Four lanes or more	75

3.5.4. Number of Design Lanes

Both AASHTO LRFD 3.6.1.1.1 and AASHTO Standard Specifications 3.6 state that a design lane should be taken as a minimum of 12 feet. The number of design lanes is the integral portion of the width of the road between the curbs divided by 12. However, when the roadway width is between 20 and 24, the number of design lanes is the integral part of the width between the curbs divided by 10, where 10 feet is the minimum width.

3.5.5. Skew Angles

Another variable to be addressed is the skew of bridges. According to AASHTO Standard Specs 3.2.6 "When a bridge is skewed, the loads and forces carried by the bridge through the deck system to pin connections and hangers should be resolved into vertical, lateral, and longitudinal force components to be considered in the design".

AASHTO LRFD defines the skew angles as "the angle between the axis of support relative to a line normal to the longitudinal axis of the bridge, i.e. a zero-degree skew denotes a rectangular bridge". Furthermore, Table 4.6.2.2.3e-1 defines a correction factor to the load distribution factor for skewed bridges. The correction factor for steel bridges is as follows:

Equation 23

$$1 - c_1(tan\ \theta)^{1.5}$$

Where:

$$c_1 = 0.25\left(\frac{K_g}{12 * L * t_s^3}\right)^{0.25}\left(\frac{S}{L}\right)^{0.5} \qquad \textit{Equation 24}$$

For $\leq \theta \leq 60$

If $\theta < 30, then\ c1 = 0$

If $\theta > 60, then\ \theta = 60$

Moreover, according to AASHTO LRFD Table 4.6.2.2.3e-1, the correction factor for the live load distribution factor for shear is:

$$1 + 0.2 * \left(\frac{12 * L * t_s^3}{K_g}\right)^{0.3} tan\ \theta \qquad \textit{Equation 25}$$

Because AASHTO LRFD provides a correction factor to deal with skew, the fact that the AASHTO Standard Specifications requires detailed calculation, and the fact that LRFD came out after the Standard Specifications, the LRFD correction factors will be assumed true for both cases. That is because they are believed to be accurate enough, and that more specific calculations are not needed to adjust for skew.

3.6. Types of Load Ratings

As previously mentioned, there are three types of load ratings, per the MBE: the design load rating, the legal load rating, and the permit load rating. The sections below will touch on the differences between them. Appendix C is a flowchart extracted from the MBE, that illustrates when the need arises to calculate the load rating under the different various conditions and methodologies.

The Design load rating serves as an initial check to identify bridges that need to be load rated for legal loads. To pass the design load rating evaluation, the Rating Factor, RF, calculated, should be greater than or equal to 1.0. If the rating factor of the design load was less than 1.0, it is possible to use the legal load loads for further investigation. Under the legal load rating method, a single safe load capacity obtained for a given truck configuration is obtained.

When selecting permits for vehicles above the legally established weight to drive on a bridge, the permit load rating procedure is applied. This check should only be applied to bridges having enough capacity under the AASHTO legal loads. For the purpose of this study, the permit load ratings are not calculated. The main purpose of this study is to conclude if truck platoons will overstress bridges. Permit load ratting are specialized and allows specific trucks to travel on a specific bridge for a limited amount of time, after they apply for a permit. Since each truck within the platoon is not overweight, they will legally be able to travel on bridges without applying for a permit. Therefore, the results obtained from the design and legal load ratings are what is needed for this study.

3.7. Load and Resistance Factor Rating (LRFR)

Bridge evacuations performed under the LRFR method utilize different live load models and evaluation criteria to yield three types of load ratings: design load rating, legal load rating, and permit load rating. Each of which serves a specific use and guides the need for further evaluations to ensure bridge safety. The LRFR method is used to measure the performance of bridges designed for the LRFD design standards.

The evaluation of the $\varphi_c, \varphi_s, and\ \varphi$ factors used in the rating factor equation are addressed in this section.

For the strength Limit States:

$$C = \varphi_c \varphi_s \varphi R_n$$

Where the following lower limit shall apply:

$$\varphi_c \varphi_s \geq 0.85$$

Per LRFD 6.5.4.2, the resistance factor, φ, shall be taken as 1.0 for flexure and shear. Since only shear and flexure are examined in this study, φ for the LRFR will be set as 1.0.

Per MBE 6A.4.2.3, the condition factor, φ_c, is dependent on the structure condition of the member. The structure condition of the member is determined based on the National Bridge Inventory (NBI) Item number 59, shown in Table 5.

Table 5: Equivalent Member Structural Condition Based on NBI Item 59, from MBE Table C6A.4.2.3-1

Superstructure Condition Rating (SI & A Item 59)	Equivalent Member Structural Condition
6 or higher	Good or Satisfactory
5	Fair
4 or lower	Poor

After determining the equivalent member structural condition based on NBI Item 59, the MBE provides Table 6 to estimate a condition factor, φ_c.

Table 6: Condition Factor Approximation, MBE Table C6A4.2.3-1

Structural Condition of Member	$\emptyset_c$
Good or Satisfactory	1.00
Fair	0.95
Poor	0.85

Per MBE 6A4.2.4, the system factor, φ_s, is extracted from Table 6A.4.3.4-1, shown in Table 7, and is based on the superstructure type.

Table 7: System Factor, MBE Table 6A.4.2.4-1

Structure Type	$\emptyset_s$
Welded Members in Two-Girder/Truss/Arch Bridges	0.85
Riveted Members in Two-Girder/Truss/Arch Bridges	0.90
Multiple Eyebar Members in Truss Bridges	0.90
Three-Girder Bridges with Girder Spacing 6 ft	0.85
Four-Girder Bridges with Girder Spacing ≤ 4 ft	0.95
All Other Girder Bridges and Slab Bridges	1.00
Floorbeams with Spacing > 12 ft and Noncontinuous Stringers	0.85
Redundant Stringer Subsystems between Floorbeams	1.00

Following the provisions in LRFD Design Appendix 6.1, the plastic neutral axis (PNA) of the section is located. Knowing where the PNA lies, the nominal moment capacity, M_n, and the nominal shear capacity, V_n, of the section are calculated.

3.7.1. Design Load Rating

The Design Load Rating is evaluated under the Strength I limit state, and the Service II limit state, when using the LRFR method. The Design Load Rating is calculated by using the AASHTO truck loads for the live loads. Under the LRFR, the HL-93 truck configuration is used. In each of the two limit states mentioned, the RF will be calculated based on the inventory and operating levels. The MBE presents a table that summarizes the load factors for load rating under the various limit states. The portion pertaining to steel sections is shown in Table 8. Under the Strength I limit, the capacities and demands are expressed as moments and shears, therefore two rating factors are evaluated per member, one of moment, and one or shear. However, under the Service II limit state, the capacities and demands are represented as stresses, and therefore only one rating factor is obtained per member.

Table 8: Load Factors for Load Ratings Under Different Limit States, MBE Table 6A.4.2.2-1

Bridge Type	Limit State*	Dead Load γ_{DC}	Dead Load γ_{DW}	Design Load Inventory γ_{LL}	Design Load Operating γ_{LL}
Steel	Strength I	1.25	1.50	1.75	1.35
	Strength II	1.25	1.50	—	—
	Service II	1.00	1.00	1.30	1.00
	Fatigue	0.00	0.00	0.75	—

3.7.2. Legal Load Rating

When the inventory load rating is less than 1.0, the legal load rating should be computed for further investigation. Similar to the design load rating, the legal load rating is studied under the Strength I limit state with shear and flexure capacities ad demands used. They are also developed under the Service II limit state using the capacity and demand stresses.

The live load distribution factors, $g_{M,I}, g_{M,E}, g_{V,I}, g_{V,E}$, are the same values calculated for the design load rating. The dynamic amplification (IM) shall be 33 percent for strength and service limit states to account for the dynamic effects due to moving vehicles. However, the dynamic amplification could be adjusted based on a field evaluation of the approach and bridge riding surface conditions. Table C6A.4.4.3-1 for the MBE presents the adjusted value of IM based on the riding surface conditions and is shown in Table 9. However, for consistency and being conservative, the dynamic impact will be the same as the IM factor used for the design truck.

Table 9: Dynamic Load Allowance for Legal Load Rating, MBE C6A.4.4.3-1

Riding Surface Conditions	IM
Smooth riding surface approaches, bridge deck, and expansion joints	10%
Minor surface deviations or depressions	25%

In the legal load rating under LRFR, the live load is obtained from AASHTO Legal Loads. These legal loads are: Type 3, Type 3S2, Type 3-3, SU4, SU5, SU6, SU7, NRL. Types SU4, SU5, SU6, and SU7 are referred to as Specialized Hauling Units (SHU), and are defined by the MBE as short wheelbase multi-axle trucks used in construction, waste management, bulk cargo and commodities hauling industries. Figure 12, Figure 13, and Figure 14 are found in the MBE Chapter 6. They illustrate the axle weights and their positions with respect to each other than make up each of the Routine Commercial Trucks and the Specialized Hauling Units, and the Notional Rating Load.

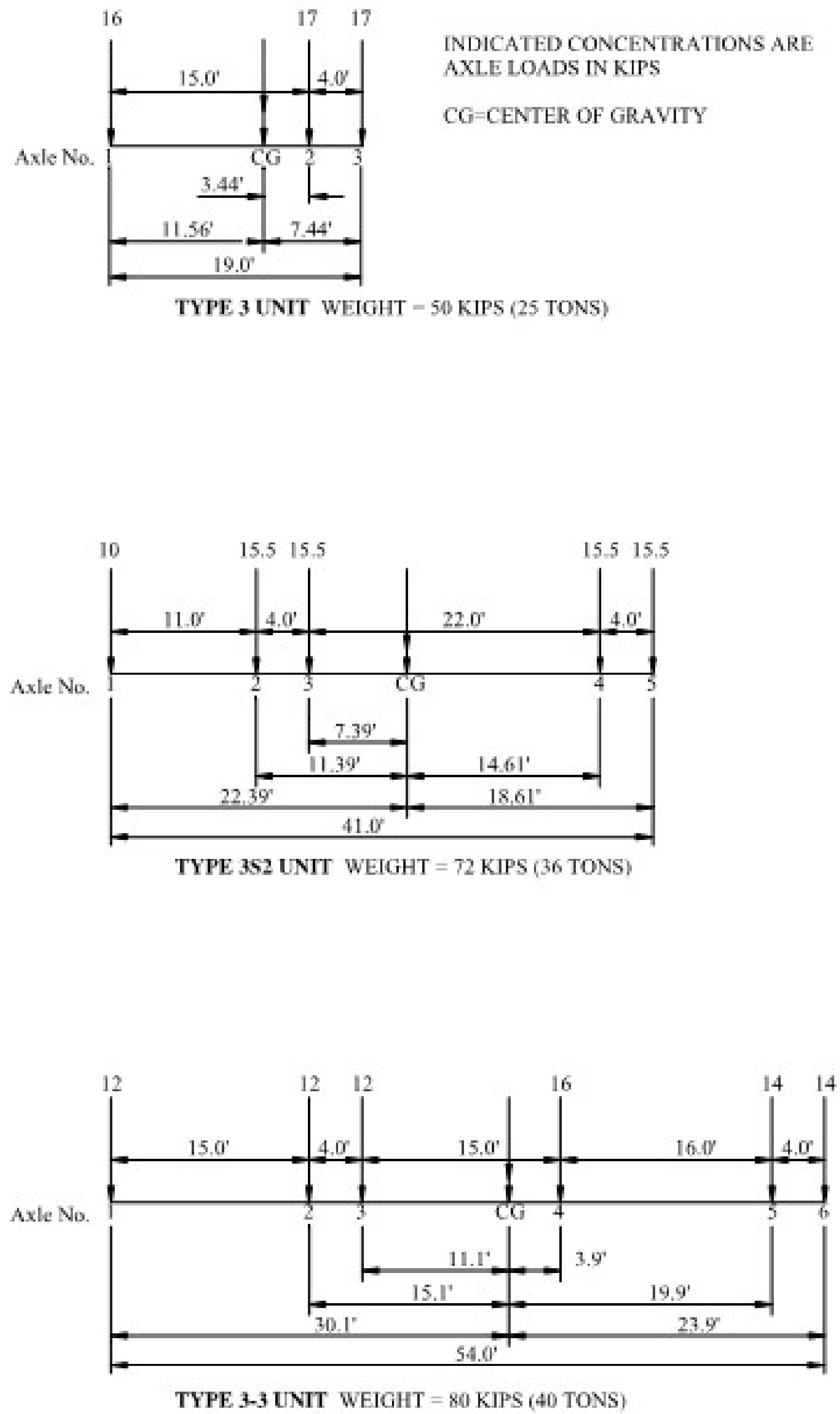


Figure 12: Routine Commercial Trucks MBE 6B.7.2-1

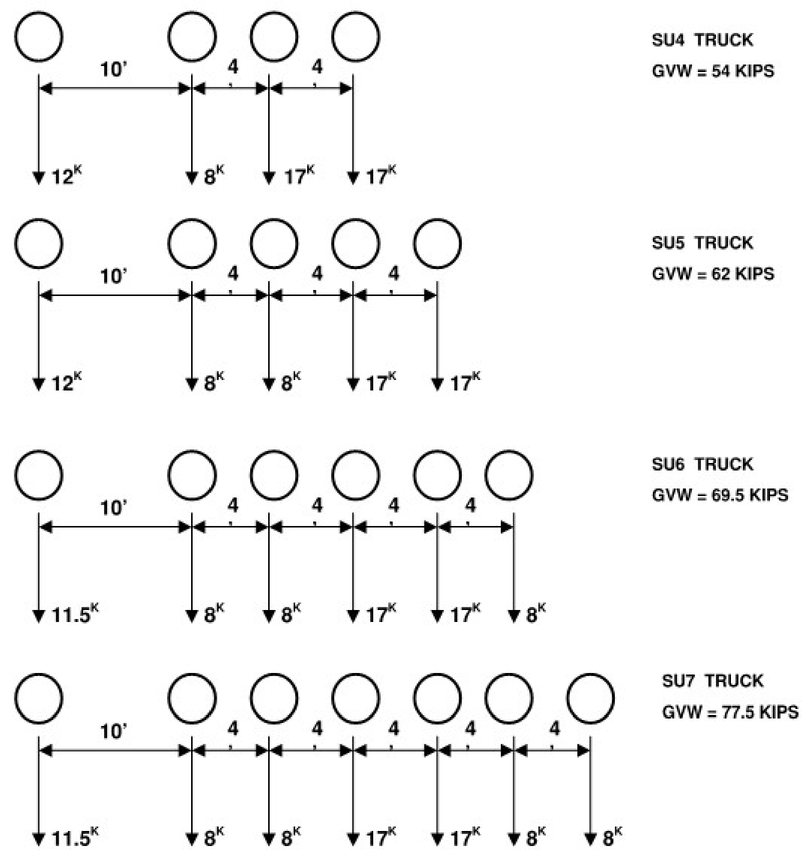
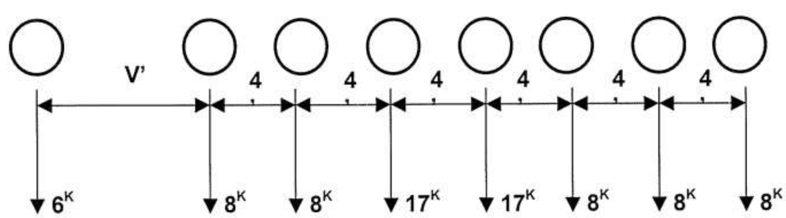


Figure 13: Specialized Hauling Units, MBE 6B.7.2-2



V = VARIABLE DRIVE AXLE SPACING –– 6'0" TO 14'-0". SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM LOAD EFFECTS.

AXLES THAT DO NOT CONTRIBUTE TO THE MAXIMUM LOAD EFFECT UNDER CONSIDERATION SHALL BE NEGLECTED.

MAXIMUM GVW = 80 KIPS

AXLE GAGE WIDTH = 6'-0"

Figure 14: Notional Rating Load: MBE 6B.7.2.3

These vehicles, along with the Notional Rating Load (NRL) have their own live load factors, presented in the MBE Table 6A.4.4.2.3b-1. Where ADTT is the average daily truck traffic, and linear interpolation is permitted for other ADTT, and is shown in Table 10.

Table 10: Live Load Factors for Specialized Hauling Vehicles, MBE 6A.4.4.2.3b-1

Traffic Volume (One direction)	Load Factor for NRL, SU4, SU5, SU6, and SU7
Unknown	1.60
ADTT ≥ 5000	1.60
ADTT= 1000	1.40
ADTT ≤ 100	1.15

As for the Type 3, 3S2, and 3-3 trucks, the live load factors presented in the MBE Table 6A.6.4.2.3a-1, shown in Table 11.

Table 11: Live Load Factors for Routine Commercial Trucks, MBE 6A.4.4.3a-1

Traffic Volume (One direction)	Load Factor for Type 3, Type 3S2, Type 3-3 and Lane Loads
Unknown	1.80
ADTT ≥ 5000	1.80
ADTT= 1000	1.65
ADTT ≤ 100	1.40

3.8. Allowable Stress Rating (ASR)

The Design Load Rating using the ASR method is calculated by using the AASHTO truck loads for the live loads. Under ASR, the HS-20 truck configuration is used. Both the operating inventory and operating levels are applicable. The ASR method is used to measure the performance of bridges designed for the AASHTO Standard Specification for Highway Bridges.

For the allowable stress method, the following variables in the rating formula are as follows:

A_1 =1

$A_2 = 1$

That is because the loads are not factored in the ASR. The Nominal Capacity, C, is calculated as a stress in the form of:

$$C = M = f_1 * S_x \quad \textit{Equation 26}$$

Where:

S_x = section modulus, as previously addressed in "section properties" (in^3)

f_1 = allowable stress capacity (ksi)

MBE Tables 6.5.2.1-1 and Tables 6.5.2.1- 2 guide the calculation of the yield strength. Appendix D includes Tables 6.5.2.1-1, used for the inventory allowable stress. Appendix E includes Tables 6.5.2.1-2, used for the operating allowable stress.

The allowable or working stress method constitutes a traditional specification to provide structural safety. The actual loadings are combined to produce a maximum stress in a member, which is not to exceed the allowable or working stress of the material and applying an appropriate factor of safety

3.9. Load Factor Rating (LFR)

The load factor rating is based on analyzing a structure to factored loads, different factors are applied to each type of loads, which reflect the uncertainty inherent in the load calculations, the rating is determined such that the effect of the factored loads does not exceed the strength of the member.

The Live load used under the LFR method is similar to the ASR method and is based on the HS-20 truck previously discussed. Under the LFR method, the inventory and operating load ratings are calculated under both the Strength and Service load cases discussed previously.

For the load factor rating, the following variables in the rating formula are as follows:

A_1 =1.3

$A_2 = 2.17$ at the Inventory Level

$A_2 = 1.3$ at the Operating Level

As the name implies, the loads are factored in the LF (Load Factor) method. The nominal capacity, C, depends on the yield strength of the steel is needed. As for the ASR method, MBE Tables 6.5.2.1-1 and Tables 6.5.2.1- 2 guide the calculation of the yield strength. Appendix D includes Tables 6.5.2.1-1, used for the inventory allowable stress. Appendix E includes Tables 6.5.2.1-2, used for the operating allowable stress.

If the member under consideration is compact, braced and non-composite, then:

$$C = F_y * Z \qquad \textit{Equation 27}$$

Where:

F_y = yield strength of the member (ksi)

$Z =$ section modulus, as previously addressed in "section properties" (in^3)

If the member under consideration is compact and composite, then:

$$C = M_u \qquad \textit{Equation 28}$$

Where:

M_u = Ultimate Moment capacity of the member (k.ft)

If the member under consideration is non-compact and non-composite, then:

$$C = S_x \qquad \textit{Equation 29}$$

Where:

S_x = section modulus, as previously addressed in "section properties " (in^3)

If the member under consideration is non-compact and composite, then:

$$C = F_y \qquad \textit{Equation 30}$$

Where:

F_y = yield strength of the member (ksi)

Refer to Appendix F for more information on the calculation of C under the LFR method.

3.10. Platoon Description

3.10.1. Truck Used

One truck was used as the typical platoon truck. Holding some variables constant is crucial for results to have significance. As a result, any changes in the resultant load ratings are a function of the bridge aspects and not the truck. The truck used is the Florida Department of Transportation C5 truck, which is a combination (C class) truck with a single trailer. Figure 15 shows the axle weights and positions of the C5 truck. Trailer (or box) trucks are the type of trucks that are expected to start platooning and driving as a fleet while transporting goods. Therefore, a C5 is a good representative truck within future platoons. Moreover, using the C5 will allow comparison to the studies by Devault (2017), and Yarnold & Weidner (2018), since those two studies used the C5 truck. The truck platoons are treated like legal loads. Therefore, all subsequent factors that will be affected, like the IM, distribution factors, load factors, etc., are calculated the using same method as the legal loads (FDOT, 2010).

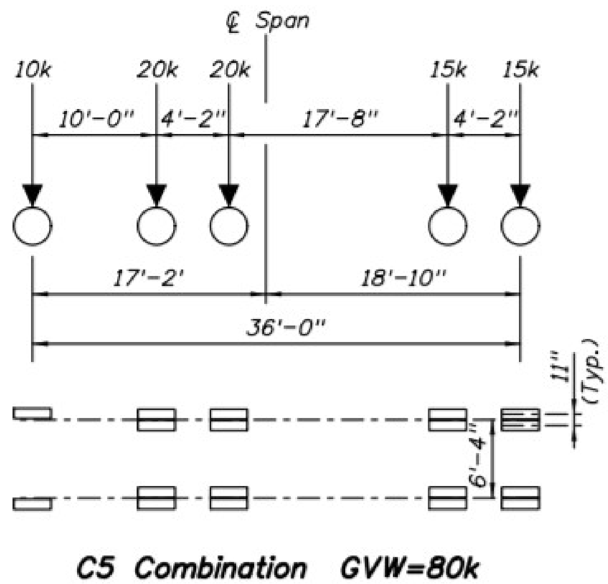


Figure 15: C5 Truck - FDOT.gov

3.10.2. Truck Spacing

The aim of this study is to generate a general understanding of the effects of platooning on bridge load ratings. Less than 20 ft spacing will not be addressed in this study because the literature indicates 20 ft as the minimum being considered. Truck spacing further than 40 ft axle to axle is also not be addressed, as that will reduce the shield that in turn reduces the air drag and thus the fuel efficiency will be reduced. In addition, Yarnold and Weidner. (2018) found that great than 40 ft spacing between trucks had relatively small impact on the live load demands.

The two following distances were considered in this study, the first being 20 ft axle-to-axle distance, which is equivalent to about 10 ft clear bumper-to-bumper distance, illustrated in Figure 16. The second is a 20 ft axle-to-axle distance which is equivalent to 10 ft clear bumper-to-bumper distance, illustrated in Figure 17. In each platoon configuration examined, the spacing is constant. For instance, all three spacings between a four-truck platoon would wither be 20 ft or 40 ft Moreover, two, three, and four-truck platoons were examined, each of the four configurations at each of the two-truck distances mentioned.

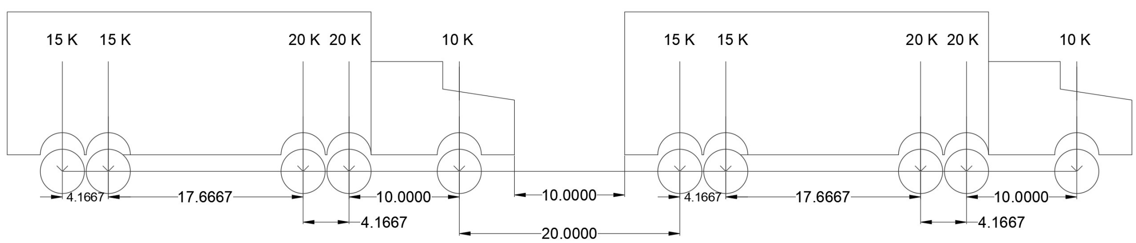


Figure 16: 20 ft Axle Spacing Platoon

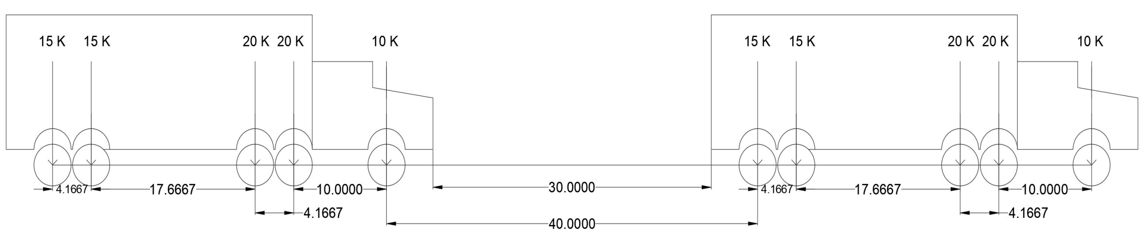


Figure 17: 40 ft Axle Spacing Platoon

3.11. Bridge Formula

Federal law states that two or more consecutive axles may not exceed the weight computed by the Bridge Formula even though single axles, tandem axles, and gross vehicle weights are within legal limits. The Bridge Formula Weights equation regulates the maximum weight that is allowed by a motor vehicle on the Interstate highway system. The United States Congress endorsed the Bridge Formula in 1975 to limit the weight-to-length ratio of a vehicle crossing a bridge, by dispersing weight over additional axles or by increasing the distance between axles. The weight limit that is determined by the Bridge Formula is shown below (U.S. Department of Transportation, 2018).

$$W = 500\ (\frac{LN}{N} + 12N + 36) \qquad \textit{Equation 31}$$

Where:

$W =$ gross weight on any group of consecutive axles to the nearest 500 lbs

$L = distance\ between\ the\ outer\ axels\ of\ any\ group\ of\ consecutive\ axels\ (ft)$

$N = number\ of\ axels\ in\ the\ group\ under\ consideration$

For comparison, the maximum weight determined by the Bridge Formula is calculated and divided by the actual weight. The configuration is accepted if the ratio is less than or equal to 1.0. The C5 truck was analyzed with the platoons (two-truck, three-truck, and four-truck, at an axle spacing of 20 ft and 40 ft) using the Bridge Formula. What the results reveal is that the 1-truck configuration controls. Figure 18 graphically represents the effect of analyzing different numbers of axles of the C5 truck on the weight of the actual to the allowable ratio. Figure 18 shows that for the C5 truck configuration, the two 20 kips axles at 4'-2" apart control the Bridge Formula check. Refer to Appendix G for the detailed results. Furthermore, the trendline plotted strengthen the argument that as the number of axles being considered, N, increase, the ratio of the actual weight to the allowable weight decreases. Therefore, the bridge formula is not likely to impose any restrictions on truck platoons.

An important note is that the C5 configuration does not meet the restrictions imposed by the Bridge Formula. The controlling 20 kips axles placed at 4'-2" apart yield a ratio of 1.17. It is also central to mention that more than 200 cases were checked for the C5 configuration using the Bridge Formula, refer to Appendix G, and the controlling case was the only one to surpass 1.0. The C5 truck is accepted by the Federal U.S. Department of transportation, regardless of the 1.17 ratio. Many states have accepted trucks that violate the Bridge Formula and have the potential to be utilized with platooning technology.

In summary, the bridge formula does not increase restrictions when platooning of the C5 truck occurs. This finding further emphasizes the importance of the results of this study.

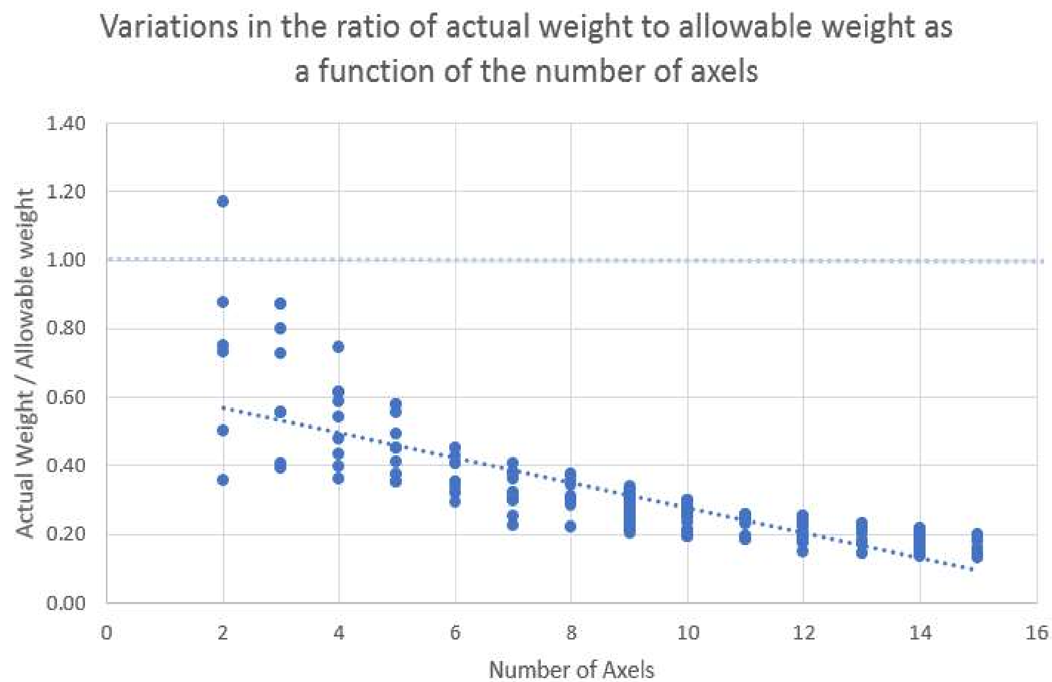


Figure 18: Variations in the Ratio of Actual Weights to Allowable Weights as a Function of Number of Axles

3.12. Bridge Configuration

The bridges selected to evaluate needed to have characteristics within the scope of this study, and have those characteristics vary is a controlled manner to facilitate correlation. Therefore, Example A1 from the MBE was used as a benchmark. Example A1 is a 65 ft single-span composite steel stringer bridge with four girders spaced at 7'- 4". With that benchmark, some parameters were changed: the span length, the number of spans, and the girder spacing. The aim was to have a single-span, a two-span, and a three-span bridge. The two-span bridges had equal span length, and the three-span bridges had the end spans equal to 80% of the length of their middle span, as illustrated on Figure 19.

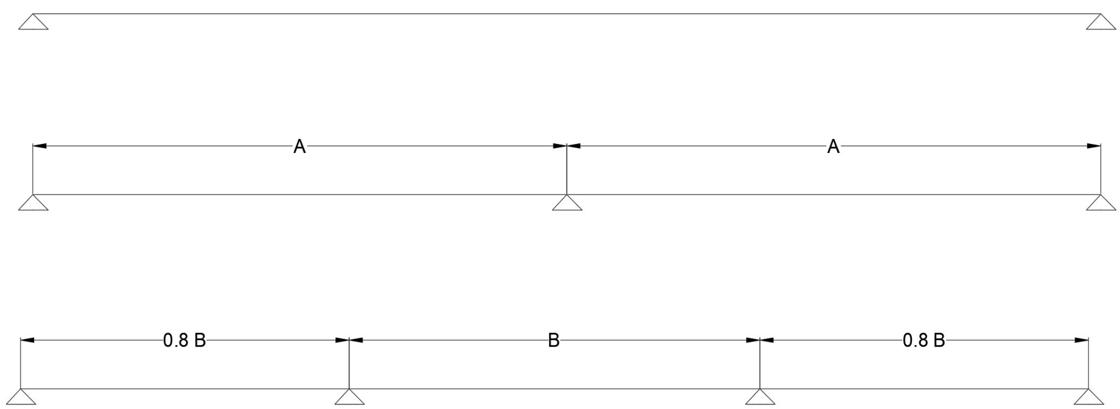


Figure 19: One, Two, and Three-Span Configurations

For each of the three-span configurations, the girder spacing was held constant, but the length of the bridge was varied to study the effect of the load rating as a function of length. Four lengths were used: 20 ft, 65 ft, 120 ft, and 240 ft. Similarly, a separate study was performed where the length was then held constant and the girder spacing was varied to study the effect of the load rating as a function of girder spacing. The girder spacings studies were: 4 ft, 8ft, 12 ft, and 16 ft.

The trucks placed on those configurations were as follows. A C-5 with a spacing of 20 ft between the rear axle of the front truck and the front axle of the rear truck was used for two, three, and four-truck platoons. Also, a C-5 with a spacing of 40' between truck axles was used for two, three, and four-truck platoons.

As the length of the bridge increased, the bridge girder size was redesigned to maintain a load rating of at least 1.0 for the legal load rating. As the spacing of the girder increased, the bridge was redesigned to also maintain a load rating of at least 1.0. The Texas

Department of transportation provides guidelines on redesigning plate girders in their "Preferred Practices for Steel Bridge Design, Fabrication, and Erection" Publication. With that, the flange width of the built-up plate girders was taken as the minimum between 15" and a fifth of the web depth, with increments in whole inches. In addition, the girder length to flange width ratio was kept under 85. The flange thickness satisfied the AASHTO b/t (base/thickness) requirements, with the minimum thickness being ¾" and the maximum being 3". The increments for the flange thickness was taken as ¼" for thicknesses between 1" and 3", and ½" for thickness between 3" and 4". Moreover, the superstructure was to have a total section depth (slab plus girder) in the range of 3.3% of the span length and 4% of the span length. Web depths were used in whole increments. The web thicknesses were taken as a minimum of 12" (TxDot, 2015).

Also, as the girder pacing increased the deck thickness was adjusted to meet guidelines of the typical deck thickness for each group of girder-spacings. The California Department of Transportation (Caltrans) provides a table that guides the determination of an appropriate deck thickness based on the girder to girder spacing. Refer to Appendix H for the Caltrans Table of Deck Slab Thickness (Caltrans, 2008).

4. RESULTS

This section presents truck platoon load rating variability with respect to the conventional AASHTO loading ratings. The first section illustrates the findings from varying the bridge span length, while holding all other parameters constant. Similarly, the second section reports the findings from varying the girder spacing, while holding all other parameters constant. The load ratings obtained when varying the span length, and varying the girder spacing, were both evaluated under the LRF, ASR, and LRFR methods.

4.1. Span Length Study

The study aimed to quantify the effects of truck platoons on bridge load ratings under varying span lengths. This was performed by comparing the truck platoon load ratings with the design and legal load ratings. For that reason, the ratio of the load rating obtained from the design to the load rating obtained from the platoons is examined. Similarly, the load rating obtained from the legal to the load rating obtained from the platoons is examined.

When calculating the load rating, one of three main categories controls: the load rating calculated using the positive moment values, the load rating calculated using the negative moment values, and the load rating calculated using the shear values. To better understand the effect of the change in span length on each of these criteria, the load rating due to each of these parameters is documented separately.

4.1.1. Positive Moment

To begin with, the effect of the span length on the load rating obtained from the positive moment effect is addressed. The design and legal load ratings are utilized as benchmarks. Figure 20, Figure 22, and Figure 24 present the variation in the ratio of the design load rating to the platoon load rating using the LFR, ASR, and LRFR methodologies, respectively. In addition Figure 21, Figure 23, and Figure 25 present the variation in the ratio of the legal load rating to the platoon load rating using the LFR, ASR, and LRFR methodologies, respectively.

As a reference for all load rating plots, the lines in solid represent the results for single-span bridges, the dotted lines are the results of a two-span bridge of equal length, and the dashed lines show the results of three-span bridges where the outside span lengths are 80% of the interior span length. For reference, the plotted length of the three-span bridges in the middle span, which is the maximum span length of the three spans. Also, each platoon configuration is represented by the same color for comparison. For example, take the color orange in the figures below. All orange lines represent a platoon of two-trucks at 20 ft axle to axle spacing. The solid orange line represents single-spans, the dotted orange line represents a two-span configuration. Lastly, the dashed orange line represents the analysis of that platoon under three-span configurations.

It is shown in Figure 20 that up to about a span length of 78 ft, the response of the AASHTO Standard Specifications live loads is close or greater than the actual response of the platoons studied, on all three bridge configurations. This is shown by the fact that the design load rating to the platoon load rating ratio remains greater than one for spans less than about 78 ft. That is explained by the fact that the C5 front axle to back axle distance is 35 ft. With that, even with the shorter platoon distance of 20 ft, two complete trucks would not fit in a span of 75 ft. Beyond the 78 ft, the ratio of the trucks spaced at the closer distance of 20 ft between their axles starts to steadily decrease. This decrease is similar between one, two, and three-span configurations. Beyond the 78 ft, the ratio of the trucks spaced at the further distance of 40 ft between their axles starts decreasing steadily. However, they decrease at a lower rate than the decrease of the 20 ft distance platoons. This decrease is also similar between single, two, and three-span configurations. Therefore, the AASHTO Standard Specifications live loads are conservative compared to the other platoons until a span of about 78 ft or less. Table 12 summarizes the findings.

For the single-span, two, three, and four truck platoons have a ratio greater than one for spans less than 78 ft for a truck distance of 20 ft, and a ratio greater than one for span length less than 108 for a truck distance of 40 ft. Similarly, for the two-span, all platoons at a 20 ft distance have a load rating greater than one for spans less than 80 ft, and spans less than 130 ft for the truck distance of 40 ft. However, for the three-span configuration, the span lengths vary as the number of trucks in the platoon changes. Therefore, load rating using the AASHTO live loads for the positive moment under the LFR methodologies is conservative up to a span length of 78 ft. Also, the two-span, and three-span responses are more conservative than the single-span. That might be associated with the fact that the AASHTO Standard Specifications live loads impose additional loading when considering continuous spans versus considering single-spans. The spans where the plotted ratio drops below one are listed in Table 12.

Table 12: LFR Positive Moment Summary, Design LR

LFR			
Configuration			Code not adequate on
# Spans	# Trucks	Distance	M +
1	2	20 ft	Span greater than 78 ft
1	2	40 ft	Spans greater than 108 ft
1	3	20 ft	Span greater than 78 ft
1	3	40 ft	Spans greater than 108 ft
1	4	20 ft	Span greater than 78 ft
1	4	40 ft	Spans greater than 108 ft
2	2	20 ft	Span greater than 80 ft
2	2	40 ft	Span greater than 130 ft
2	3	20 ft	Span greater than 85 ft
2	3	40 ft	Span greater than 130 ft
2	4	20 ft	Span greater than 85 ft
2	4	40 ft	Span greater than 130 ft
3	2	20 ft	Span greater than 78 ft
3	2	40 ft	Span greater than 109 ft
3	3	20 ft	Span greater than 88 ft
3	3	40 ft	Span greater than 126 ft
3	4	20 ft	Span greater than 98 ft
3	4	40 ft	Span greater than 126 ft

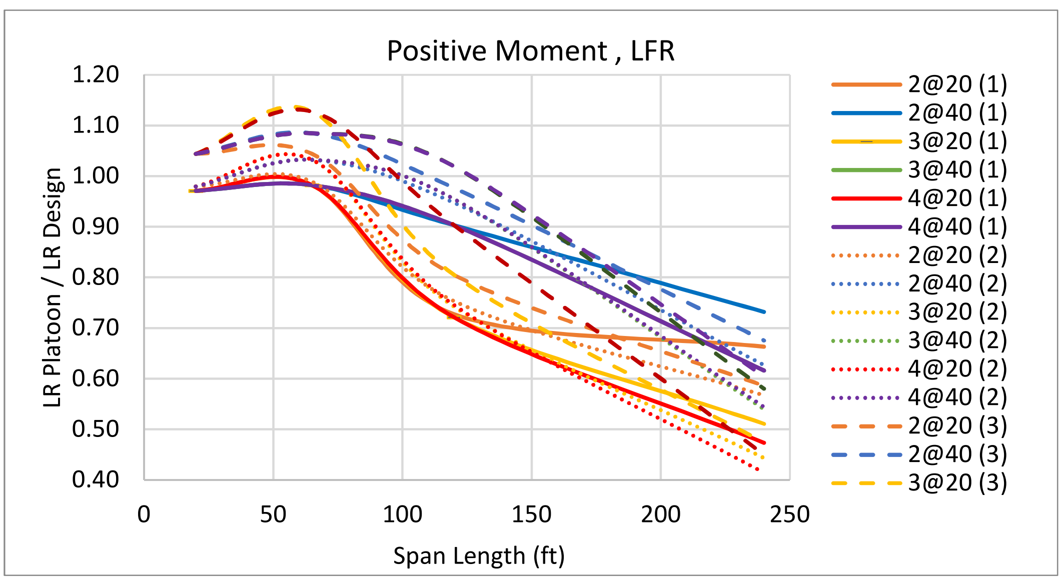


Figure 20: LR Platoon/LR Design, Positive Moment, LFR

Per the MBE load rating chart shown in Appendix C, if the design load rating is less than one, then the bridge is still acceptable if the legal load rating is greater than one. Therefore, the legal load rating is examined for each of the spans identified when looking at the design load rating ratio. Under the LFR method, the ratio of the legal load rating to that of the platoon load rating is greater than one for all examined configurations, shown in Figure 21. Table 13 lists the span lengths that have a design load rating ratio less than one, and that continue to have a ratio of less than one, even after examining their legal loads.

Table 13: LFR Positive Moment Summary, Legal LR

LFR				
Configuration			Code not adequate on (per design)	Code not adequate on (per legal)
# Spans	# Trucks	Distance	M +	M +
1	2	20 ft	Span greater than 78 ft	None
1	2	40 ft	Spans greater than 108 ft	None
1	3	20 ft	Span greater than 78 ft	None
1	3	40 ft	Spans greater than 108 ft	None
1	4	20 ft	Span greater than 78 ft	None
1	4	40 ft	Spans greater than 108 ft	None
2	2	20 ft	Span greater than 80 ft	None
2	2	40 ft	Span greater than 130 ft	None
2	3	20 ft	Span greater than 85 ft	None
2	3	40 ft	Span greater than 130 ft	None
2	4	20 ft	Span greater than 85 ft	None
2	4	40 ft	Span greater than 130 ft	None
3	2	20 ft	Span greater than 78 ft	None
3	2	40 ft	Span greater than 109 ft	None
3	3	20 ft	Span greater than 88 ft	None
3	3	40 ft	Span greater than 126 ft	None
3	4	20 ft	Span greater than 98 ft	None
3	4	40 ft	Span greater than 126 ft	None

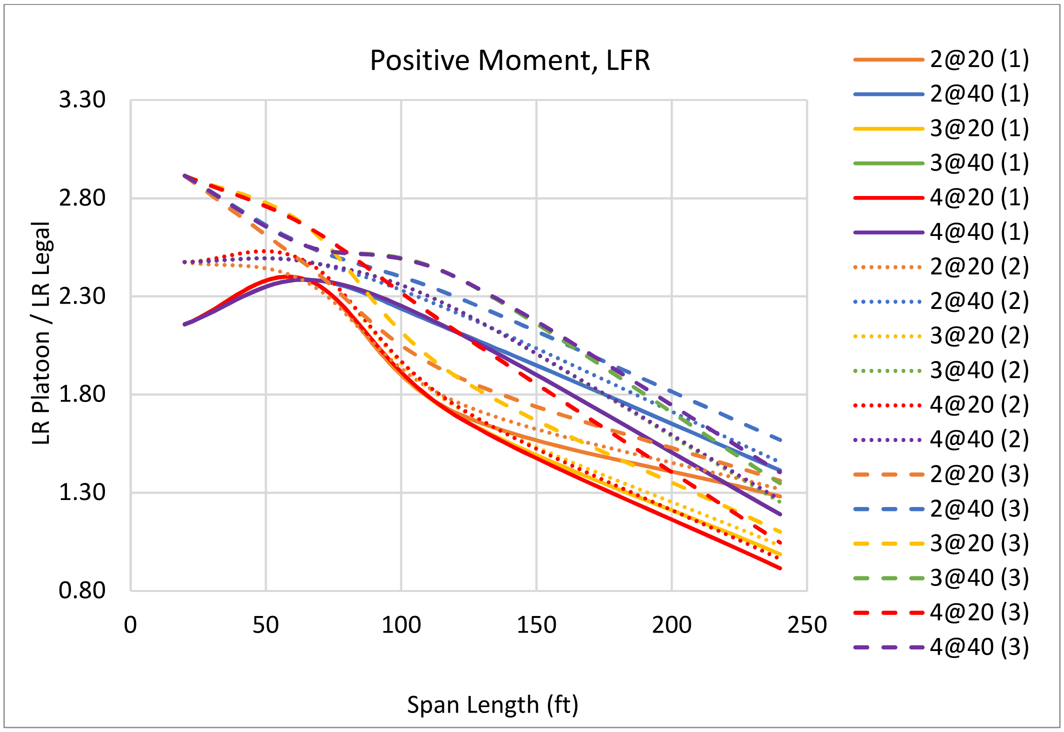


Figure 21: LR Platoon/LR Legal, Positive Moment, LFR

As previously stated, the output from the LFR and ASR rating methods is very similar for the design load rating. However, under LFR, single-spans had a ratio of one or greater up to a span length of 78 ft. Under the ASR methodology, all single-span configurations have a ratio less than one. That might seem counter-intuitive, but it is associated with the fact that the AASHTO Standard Specifications live loads actually impose additional loading when considering continuous spans versus considering single-spans.

A few trends to note are as follows. For the two-span configuration, two-truck and three-truck platoons at 40 ft have their plots almost superimposed. Therefore, the additional third truck, in that case, is not making a difference in the load demand. They both have a ratio greater than one up to a span length of 100 ft. The same can be said for the two-span configuration, two-truck and three-truck platoons at a distance of 20 ft. However, they retain a ratio of greater than one up to a span length of 75 ft. For the three-span configuration, only the two-truck and three-truck platoons at a distance of 40 converge and have a ratio greater than one out to a span length of 127 ft. Table 14 summarizes the findings.

The ratio of the trucks spaced at the distance of 40 ft between their axles decreases steadily. However, they decrease at a lower rate than the decrease of the 20 ft distance platoons. This decrease is also similar between single, two, and three-span configurations. Therefore, load rating using the AASHTO live loads for the positive moment under the LFR methodologies is conservative up to a span length of 63 ft on two and three-span bridges. The spans where the plotted ratio drops below one are documented in Figure 13.

Table 14: ASR Positive Moment Summary, Design LR

ASR			
Configuration			Code not adequate on
# Spans	# Trucks	Distance	M +
1	2	20 ft	All Spans
1	2	40 ft	All Spans
1	3	20 ft	All Spans
1	3	40 ft	All Spans
1	4	20 ft	All Spans
1	4	40 ft	All Spans
2	2	20 ft	Span greater than 63 ft
2	2	40 ft	Span greater than 95 ft
2	3	20 ft	Span greater than 75 ft
2	3	40 ft	Span greater than 100 ft
2	4	20 ft	Span greater than 75 ft
2	4	40 ft	Span greater than 100 ft
3	2	20 ft	Span greater than 76 ft
3	2	40 ft	Span greater than 108 ft
3	3	20 ft	Span greater than 86 ft
3	3	40 ft	Span greater than 127 ft
3	4	20 ft	Span greater than 95 ft
3	4	40 ft	Span greater than 127 ft

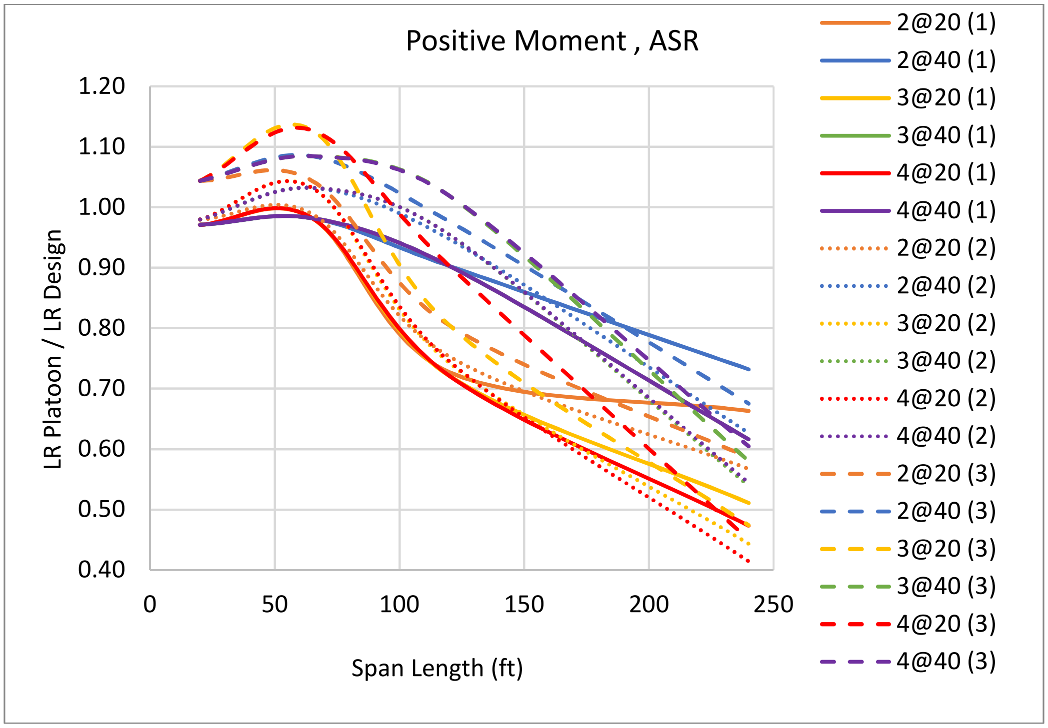


Figure 22: LR Platoon/LR Design, Positive Moment, ASR

Examining the ASR Figure 23, the legal load rating to platoon load rating ratio does not remain greater than one for all configurations.

Table *15* lists the span lengths that have a design load rating ratio less than one, and that continue to have a ratio of less than one, even after examining their legal loads.

Table 15: ASR Legal Load Rating, Positive Moment

ASR				
Configuration			Code not adequate on (per design)	Code not adequate on (per legal)
# Spans	# Trucks	Distance	M +	M +
1	2	20 ft	All Spans	Spans greater than 105 ft
1	2	40 ft	All Spans	Spans greater than 134 ft
1	3	20 ft	All Spans	Spans greater than 105 ft
1	3	40 ft	All Spans	Spans greater than 134 ft
1	4	20 ft	All Spans	Spans greater than 105 ft
1	4	40 ft	All Spans	Spans greater than 134 ft
2	2	20 ft	Span greater than 63 ft	Spans greater than 225 ft
2	2	40 ft	Span greater than 95 ft	Spans greater than 240 ft
2	3	20 ft	Span greater than 75 ft	Spans greater than 180 ft
2	3	40 ft	Span greater than 100 ft	Spans greater than 225 ft
2	4	20 ft	Span greater than 75 ft	Spans greater than 180 ft
2	4	40 ft	Span greater than 100 ft	Spans greater than 225 ft
3	2	20 ft	Span greater than 76 ft	None
3	2	40 ft	Span greater than 108 ft	None
3	3	20 ft	Span greater than 86 ft	Spans greater than 225 ft
3	3	40 ft	Span greater than 127 ft	None
3	4	20 ft	Span greater than 95 ft	Spans greater than 225 ft
3	4	40 ft	Span greater than 127 ft	None

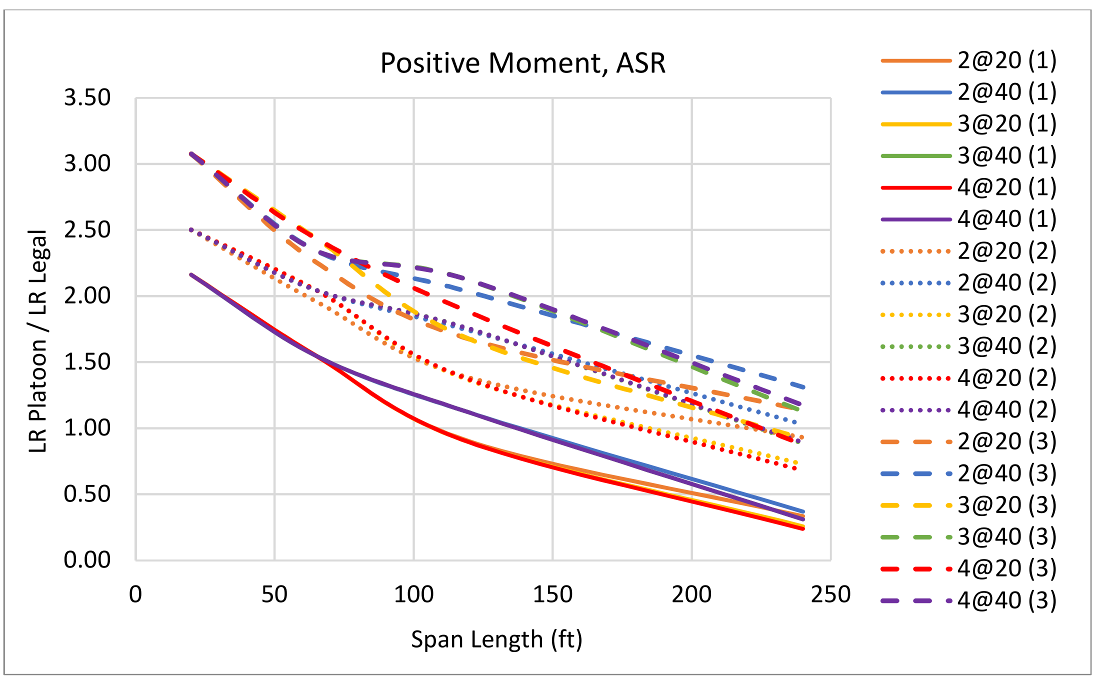


Figure 23: LR Platoon/LR Legal, Positive Moment, ASR

As for the LRFR methodology, Figure 24 shows that the response differs from that of the ASR and LFR. Under this methodology, the ratio of most configurations is decreasing steadily with an increase in span length. Some plots trend upwards. Those are the plots for the two-trucks at 20 ft for all three span configurations and the two-trucks at 40 ft for the single-span configuration. The reason behind that trend is that the effect of two-trucks at the spacing of 20 ft is less severe than the effect created by the AASHTO live load. Also, for the single-span, two-trucks at 40 ft do not impose as much load as the AASHTO live load do, therefore the trend for both the single-span and the double-span configurations, where the single-span is accompanied by a slight increase, while the two-pan undergoes a slight decrease. The ratio for truck platoons with a 40 ft axle spacing remains higher than that of the truck platoons with a 20 ft axle spacing, as they both steadily decrease with an increase in span length. The cases where the ratio of the design load rating to the platoon load rating drops below one is where attention is needed. In the LRFR methodology, the ratio drops below one and continues to steadily decrease for some configurations. To note, all platoons with an axle spacing of 40 ft, and all platoons with two-trucks output a load rating higher than that of the AASHTO live load configuration. Also, for a span length less than 140 ft the AASHTO live load produces a lower load rating than any of the configurations. Therefore, load rating using the AASHTO live loads for the positive moment under the LRFR methodology is conservative up to a span length of 140 ft. The configurations and results are summarized in Table 16.

Table 16: LRFR Positive Moment Summary, Design LR

LRFR			
Configuration			Code not adequate on
# Spans	# Trucks	Distance	M +
1	2	20 ft	None
1	2	40 ft	None
1	3	20 ft	Spans greater than 234 ft
1	3	40 ft	None
1	4	20 ft	Spans greater than 186 ft
1	4	40 ft	None
2	2	20 ft	None
2	2	40 ft	None
2	3	20 ft	Spans greater than 140 ft
2	3	40 ft	None
2	4	20 ft	Spans greater than 140 ft
2	4	40 ft	None
3	2	20 ft	None
3	2	40 ft	None
3	3	20 ft	Spans greater than 180 ft
3	3	40 ft	None
3	4	20 ft	Spans greater than 148 ft
3	4	40 ft	None

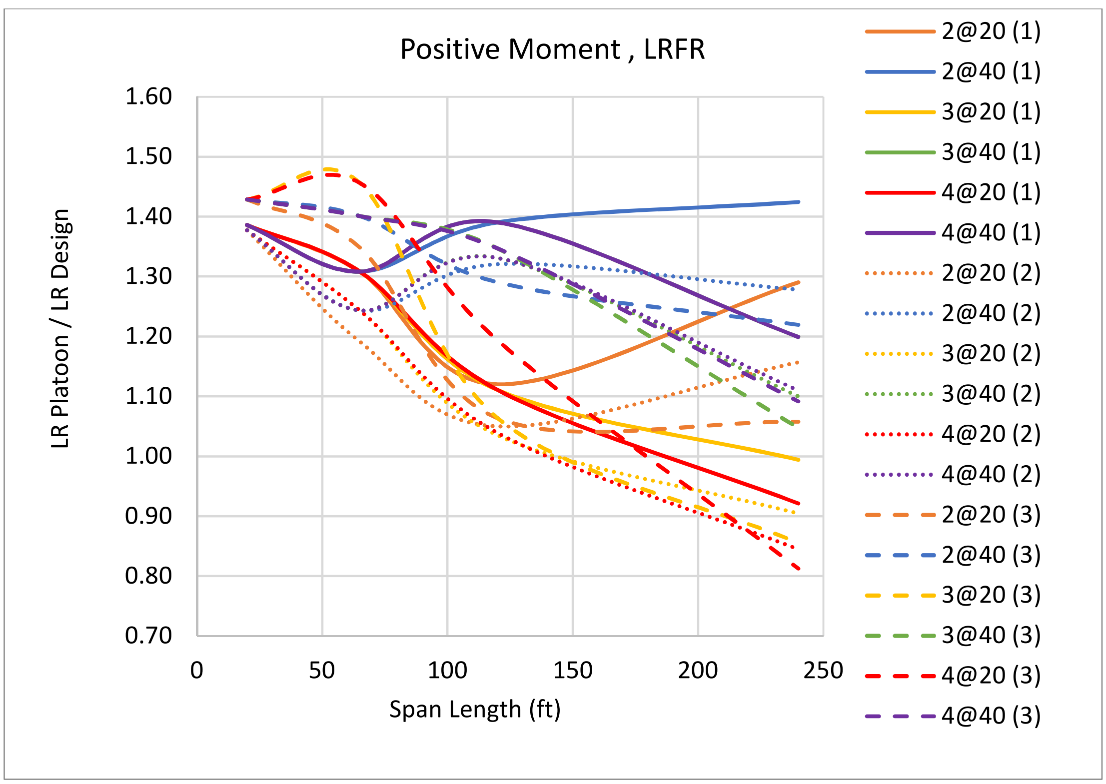


Figure 24: LR Platoon/LR Design, Positive Moment, LRFR

Examining the LRFR Figure 25**Error! Reference source not found.**, the legal load rating to platoon load rating ratio does not remain over one for all configurations. Also, after examination, the design load rating evaluation remains above one for longer spans than the legal load rating does. Because of that, the legal load rating evaluation, in this case, does not allow the increase the span length of certain configurations. For the configurations have a ratio less than one for the design load rating, the results are summarized in Table 17.

Table 17: LRFR, Legal Load Rating, Positive Moment

LRFR				
Configuration			Code not adequate on (per design)	Code not adequate on (per legal)
# Spans	# Trucks	Distance	M +	M +
1	2	20 ft	None	None
1	2	40 ft	None	None
1	3	20 ft	Spans greater than 234 ft	Spans greater than 234 ft
1	3	40 ft	None	None
1	4	20 ft	Spans greater than 186 ft	Spans greater than 186 ft
1	4	40 ft	None	None
2	2	20 ft	None	None
2	2	40 ft	None	None
2	3	20 ft	Spans greater than 140 ft	Spans greater than 140 ft
2	3	40 ft	None	None
2	4	20 ft	Spans greater than 140 ft	Spans greater than 140 ft
2	4	40 ft	None	None
3	2	20 ft	None	None
3	2	40 ft	None	None
3	3	20 ft	Spans greater than 180 ft	Spans greater than 180 ft
3	3	40 ft	None	None
3	4	20 ft	Spans greater than 148 ft	Spans greater than 148 ft
3	4	40 ft	None	None

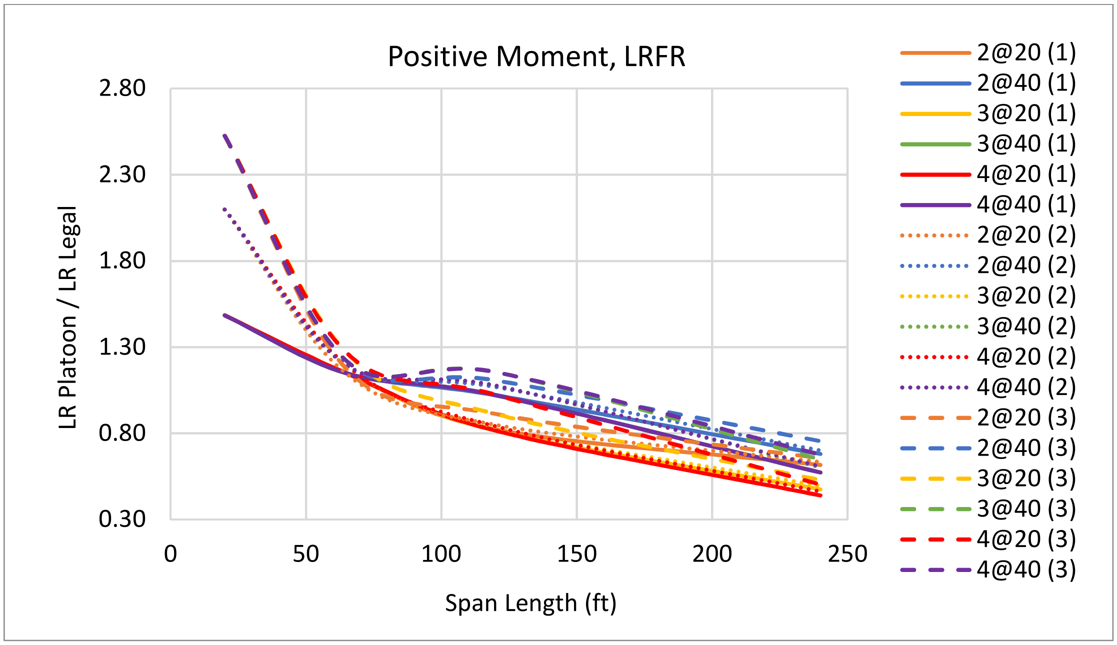


Figure 25: LR Platoon/LR Legal, Positive Moment, LRFR

4.1.2. Negative Moment

The effect of the span length on the load rating obtained from the negative moment effect is addressed next. The design and legal load ratings are utilized as benchmarks. The following Figure 26, Figure 28,and Figure 30 present the variation in the ratio of the design load rating to the platoon load rating using the LFR, ASR, and LRFR methodologies, respectively. And, the following Figure 27, Figure 29**Error! Reference source not found.**, and Figure 31 present the variation in the ratio of the legal load rating to the platoon load rating using the LFR, ASR, and LRFR methodologies, respectively. Single-spans do not have negative moment, therefore, only two and three-span configurations are addressed in this section. As a reference, the dotted lines represent the results for a two-span of equal length, and the dashed lines show the results of three-span bridges where the outside span length is 80% of the interior span length.

For all three methodologies, the trend of the ratio of the design load rating to that of the platoon appears similar. The design load rating to platoon load rating ratio decreases steadily until it reaches a minimum at a span length of around 100 ft, where it starts to increase again. However, the areas of concern are when that ratio drops below one. Even though the trend is similar, the area where the ratio drop below one is not the same for all three configurations.

For the LFR method, presented below in Figure 26, almost all truck platoons have a ratio of less than one. The exception is the two-span bridge with a two-truck platoon, spaced at both 20 ft and 40 ft, at span lengths greater than about 130 ft. However, the AASHTO live loads are not enough to cover any of the three-span configurations. The configurations that need attention are the ones where the ratio drops below one are listed in

Table 18.

Table 18: LFR Negative Moment Summary, Design LR

LFR			
Configuration			Code not adequate on
# Spans	# Trucks	Distance	M -
2	2	20 ft	Spans smaller than 110 ft
2	2	40 ft	Spans smaller than 131 ft
2	3	20 ft	All Spans
2	3	40 ft	Spans smaller than 161 ft
2	4	20 ft	Spans smaller than 212 ft
2	4	40 ft	All Spans
3	2	20 ft	All Spans
3	2	40 ft	Spans less than 200 ft
3	3	20 ft	All Spans

3	3	40 ft	All Spans
3	4	20 ft	All Spans
3	4	40 ft	All Spans

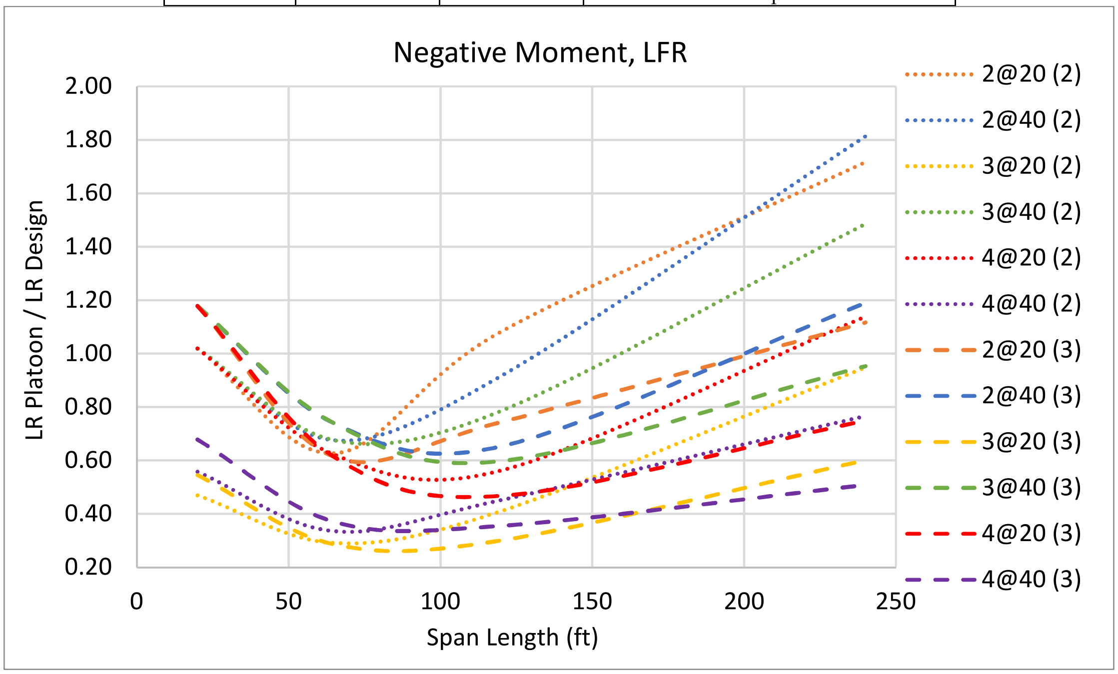


Figure 26: LR Platoon/LR Design, Negative Moment, LFR

Figure 27 shows that the legal load rating to platoon load rating ratio does not remain over one for all configurations. The configurations that did not have load rating greater than one under the design load rate are re-examined under the legal load rating benchmark. Some configurations remain more demanding than the AASHTO live loads, and therefore, require attention. Those are the two-span, three-truck platoon spaced at 20 ft, the two-span, four-truck platoon spaced at 20 ft, the two-span, four-truck platoon spaced at 40 ft, the three-span, three-truck platoon spaced at 20 ft, the three-span, four-truck platoon spaced at 20 ft, and the three-span, four-truck platoon spaced at 40 ft. Table 19 summarizes those findings.

Table 19: LFR, Legal Load Rating, Negative Moment

LFR				
Configuration			Code not adequate on (per design)	Code not adequate on (per legal)
# Spans	# Trucks	Distance	M -	M -
2	2	20 ft	Spans smaller than 110 ft	None
2	2	40 ft	Spans smaller than 131 ft	None
2	3	20 ft	All Spans	All Spans
2	3	40 ft	Spans smaller than 161 ft	Spans between 130 ft and 161 ft
2	4	20 ft	Spans smaller than 212 ft	Spans between 85 ft and 212 ft
2	4	40 ft	All Spans	All Spans
3	2	20 ft	All Spans	None
3	2	40 ft	Spans less than 200 ft	None
3	3	20 ft	All Spans	Spans greater than 37 ft
3	3	40 ft	All Spans	Spans greater than 200 ft
3	4	20 ft	All Spans	Spans greater than 90 ft
3	4	40 ft	All Spans	Spans greater than 50 ft

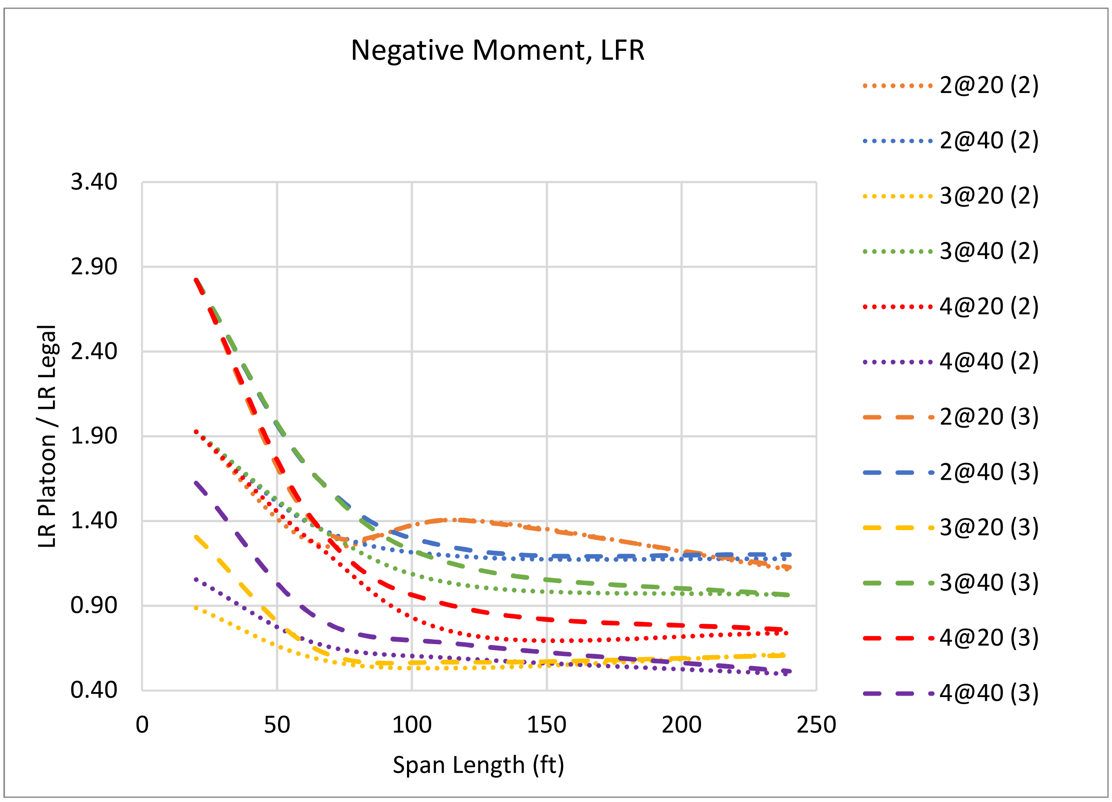


Figure 27: LR Platoon/LR Legal, Negative Moment, LFR

Examining the results from the ASR method, beyond a span length of 130 ft, all two-span truck-platoon configurations have a load rating higher than the design load rating. Therefore, at span lengths greater than 130 ft, AASHTO live loads are conservative for two-span bridges. For span length less than 130 ft, the design load rating and the platoon road rating for both, two-span and three-span bridges are very close, but the platoon remains slightly lower than the AASHTO live loads, which makes the AASHTO live loads less conservative. The configurations that need attention are the ones where the ratio drops below one are shown in Figure 28 and listed in Table 20.

Table 20: ASR Negative Moment Summary, Design LR

ASR			
Configuration			Code not adequate on
# Spans	# Trucks	Distance	M -
2	2	20 ft	Spans less than 85 ft
2	2	40 ft	Spans less than 92 ft
2	3	20 ft	Spans less than 116 ft
2	3	40 ft	Spans less than 107 ft
2	4	20 ft	Spans less than 130 ft
2	4	40 ft	Spans less than 107 ft
3	2	20 ft	Spans less than 140 ft
3	2	40 ft	Spans less than 152 ft
3	3	20 ft	Spans less than 212 ft
3	3	40 ft	Spans less than 185 ft
3	4	20 ft	All Spans
3	4	40 ft	All Spans

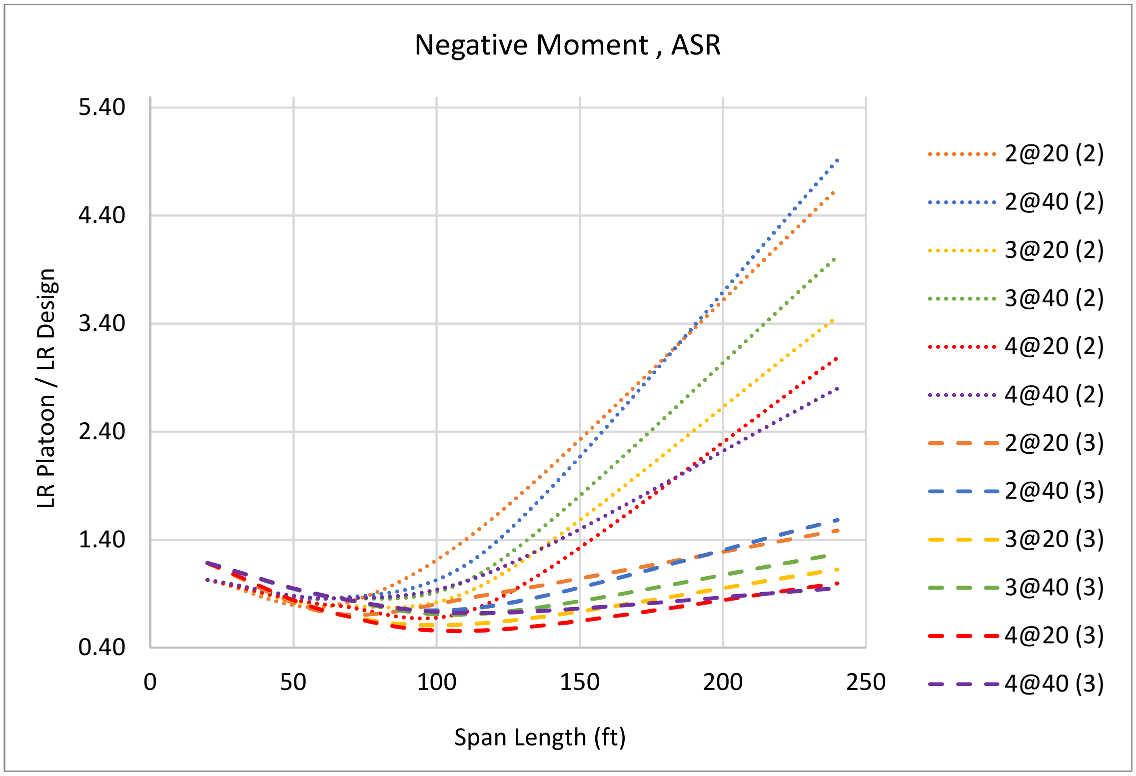


Figure 28: LR Platoon/LR Design, Negative Moment, ASR

Figure 29 shows that the legal load rating to platoon load rating ratio does not remain over one for all configurations. The configurations that did not have load rating greater than one under the design load rate are reexamined under the legal load rating benchmark. Some configurations remain more demanding than the AASHTO live loads, and therefore, require attention. Table 21Table 19 summarizes those findings.

Table 21: ASR, Legal Load Rating, Negative Moment

ASR				
Configuration			Code not adequate on (per design)	Code not adequate on (per legal)
# Spans	# Trucks	Distance	M -	M -
2	2	20 ft	Spans less than 85 ft	None
2	2	40 ft	Spans less than 92 ft	None
2	3	20 ft	Spans less than 116 ft	Spans less than 70 ft
2	3	40 ft	Spans less than 107 ft	Spans less than 80 ft
2	4	20 ft	Spans less than 130 ft	Spans less than 70 ft
2	4	40 ft	Spans less than 107 ft	Spans less than 80 ft
3	2	20 ft	Spans less than 140 ft	None
3	2	40 ft	Spans less than 152 ft	None
3	3	20 ft	Spans less than 212 ft	Spans less than 88 ft
3	3	40 ft	Spans less than 185 ft	Spans less than 120 ft
3	4	20 ft	All Spans	Spans less than 88 ft
3	4	40 ft	All Spans	Spans less than 120 ft

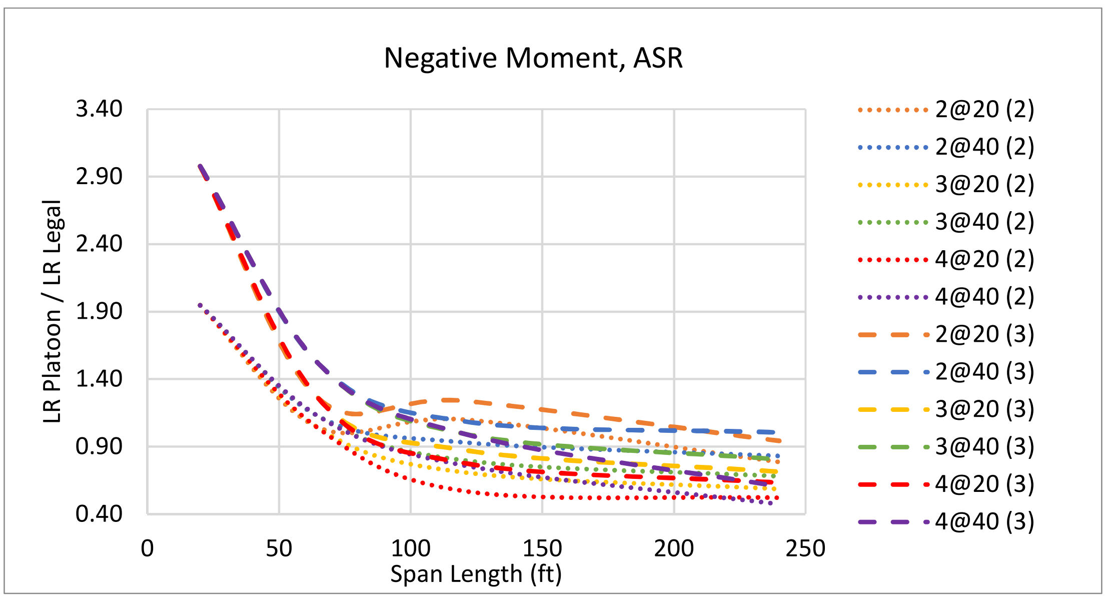


Figure 29: LR Platoon/LR Legal, Negative Moment, ASR

Figure 30 shows that for the negative moment effect under the LRFR methodology, the ratio of design load rating to that of the platoon load rating remains over 1 for both two-span, and three-span bridges, under all platoon configurations. The general trend of the ratio decreasing then increasing remains similar to the ASR and LFR methodologies. Also, the increase is steeper for two-spans than it is for three-spans. However, it is conservative to use the AASHTO live loads to load rate for the negative moment under LRFR methodology. The results are summarized in Table 22.

Table 22: LRFR Negative Moment Summary, Design LR

LRFR			
Configuration			Code not adequate on
# Spans	# Trucks	Distance	M -
2	2	20 ft	Spans
2	2	40 ft	None
2	3	20 ft	None
2	3	40 ft	None
2	4	20 ft	None
2	4	40 ft	None
3	2	20 ft	None
3	2	40 ft	None
3	3	20 ft	None
3	3	40 ft	None
3	4	20 ft	None
3	4	40 ft	None

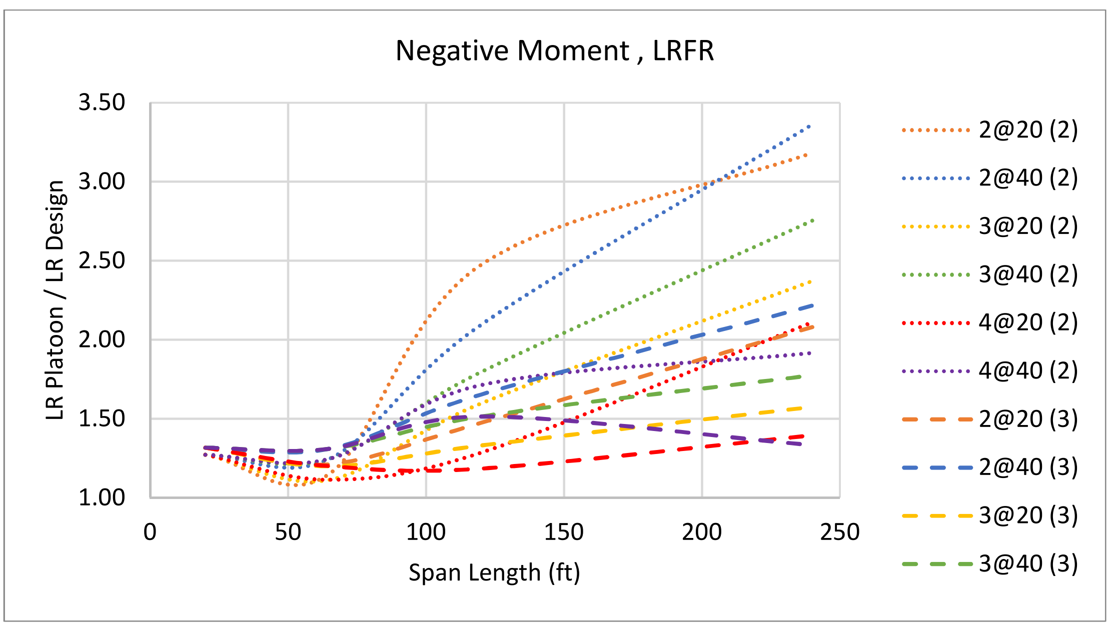


Figure 30: LR Design/LR Platoon, Negative Moment, LRFR

Figure 31 contains the legal load rating for negative moments under the LRFR methodologies. The results obtained from the design load rating will not need to be reevaluated for the legal load rating, as they are all greater than one. However, it is useful in the event a design load rating does drop below one. Table 23 shows the spans that have a load rating ratio less than one when evaluated under the design loads and after being evaluated under the legal design loads.

Table 23: LRFR, Legal Load Rating, Negative Moment

LRFR				
Configuration			Code not adequate on (per design)	Code not adequate on (per legal)
# Spans	# Trucks	Distance	M -	M -
2	2	20 ft	None	None
2	2	40 ft	None	None
2	3	20 ft	None	None
2	3	40 ft	None	None
2	4	20 ft	None	None
2	4	40 ft	None	None
3	2	20 ft	None	None
3	2	40 ft	None	None
3	3	20 ft	None	None
3	3	40 ft	None	None
3	4	20 ft	None	None
3	4	40 ft	None	None

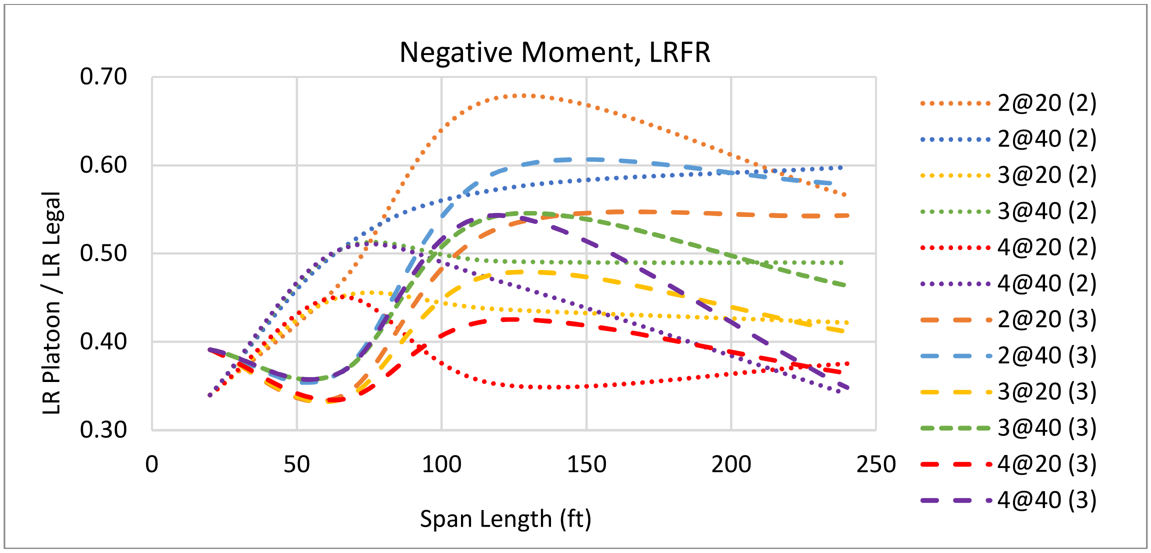


Figure 31: LR legal/LR Platoon, Negative Moment, LRFR

4.1.3. Shear

The shear capacity was checked for all the cases evaluated. However, the shear load rating was only evaluated for the LRFR cases. The reason behind that is that the MBE design examples did not calculate the shear load rating for the ASM and LFM methods. Not being able to verify the data, it was not calculated.

Figure 32 shows the design load rating to the platoon load ration ratio for shear. It is important to note that shear does not have a legal load rating. The two-truck at 40 ft distance configuration retains a ratio of greater than one under single, two, and three-span. The configurations that need attention are the ones where the ratio drops below one are listed in Table 24.

Table 24: Shear LR

LRFR, Shear			
Configuration			Code not adequate on
# Spans	# Trucks	Distance	M +
1	2	20 ft	Spans greater than 101 ft
1	2	40 ft	None
1	3	20 ft	Spans greater than 101 ft
1	3	40 ft	Spans greater than 173 ft
1	4	20 ft	Spans greater than 173 ft
1	4	40 ft	Spans greater than 215 ft
2	2	20 ft	Spans greater than 95 ft
2	2	40 ft	None
2	3	20 ft	Spans greater than 88 ft
2	3	40 ft	Spans greater than 130 ft
2	4	20 ft	Spans greater than 150 ft
2	4	40 ft	Spans greater than 183 ft
3	2	20 ft	Spans greater than 100 ft
3	2	40 ft	None
3	3	20 ft	Spans greater than 95 ft
3	3	40 ft	Spans greater than 138 ft
3	4	20 ft	Spans greater than 155 ft
3	4	40 ft	Spans greater than 198 ft

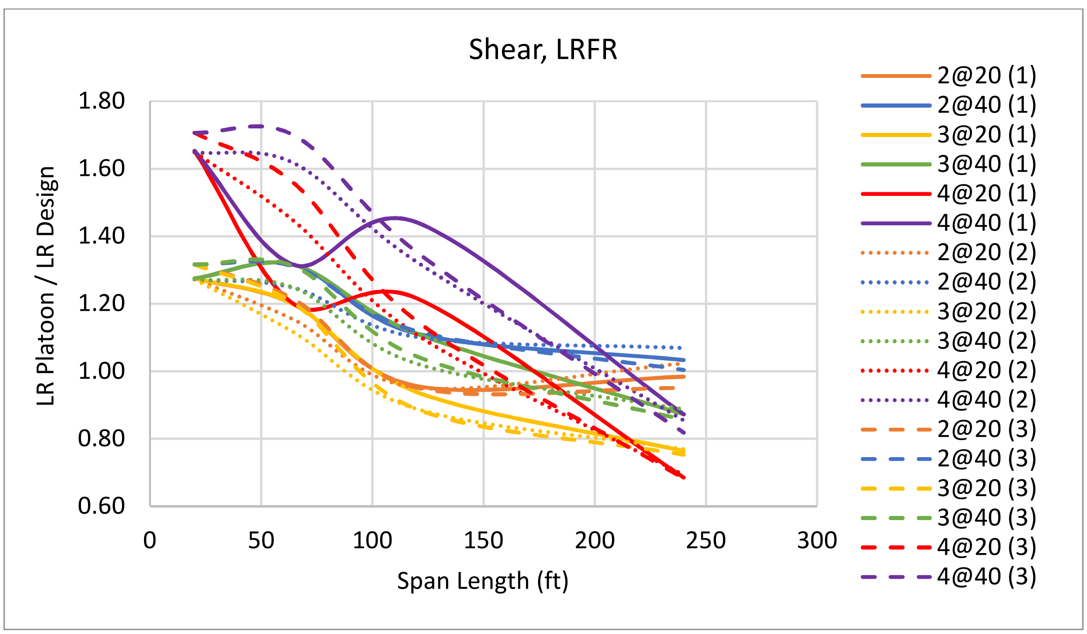


Figure 32: LR design/LR Platoon, Shear, LRFR

The Conclusions section combines all these findings together to see a big picture inference. It ceases to separate negative moment, positive moment, and instead concludes what configurations are expected to perform well under ASR, LFR, or LRFR, and what situations require additional attention to safely operate under truck platoons.

4.2. Girder Spacing Study

As previously mentioned, a similar study was conducted by varying the girder spacings and adjusting the girder size and deck thickness. That study was also performed under ASR, LFR, and LRFR methodologies, and separated the results for legal and design, positive moment, negative moment, and shear. However, examining the girder spacing output did not yield conclusive results. The results were varied and did not follow any significant trend. For those reasons, the results will not be addressed further. However, they are included in Appendix I for reference.

5. CONCLUSIONS AND FURTHER DIRECTION

As previously mentioned, the conclusions section begins by analyzing the obtained data results to draw a big picture inference about the effects of truck platoons on the load rating of steel bridges. After analysis of the results, the bridge and platoon characteristics that would require additional considerations than currently in the code are identified. Similarly, the bridge and platoon characteristics that would be conservative to evaluate as per the code's notional design trucks and loads and legal trucks are also identified. Table 25, Table 26, and Table 27 represent a summary of bridge and platoon characteristics that rate sufficiently under the current code, with the span restrictions imposed, and for the scope of parameters investigated as part of this study.

The LFR methodology results are examined next and are summarized in Table 25. Combining the results from positive and negative moment, the span restrictions on platoon and bridge configurations are as follows. All configurations can be conservatively evaluated under the current LFR methodology code without any additional restrictions with a single-span bridge. Additionally, two-truck platoons, at a distance of 20 ft or 40 ft, and three-truck platoons at a distance of 40 ft, on single-span, two-span, and three-span bridges, do not require additional considerations than what is presently in the code. Areas that require additional attention are three and four-truck platoons on two and three spans. Table 25 summarizes the results.

Table 25: LFR Configuration Summary

LFR					
Configuration			Code not adequate on		
# Spans	# Trucks	Distance	M +	M -	Result
1	2	20 ft	None	None	None
1	2	40 ft	None	None	None
1	3	20 ft	None	None	None
1	3	40 ft	None	None	None
1	4	20 ft	None	None	None
1	4	40 ft	None	None	None
2	2	20 ft	None	None	None
2	2	40 ft	None	None	None
2	3	20 ft	None	All Spans	All Spans
2	3	40 ft	None	Spans between 130 ft and 161 ft	Spans between 130 ft and 161 ft
2	4	20 ft	None	Spans between 85 ft and 212 ft	Spans between 85 ft and 212 ft
2	4	40 ft	None	All Spans	All Spans
3	2	20 ft	None	None	None
3	2	40 ft	None	None	None
3	3	20 ft	None	Spans greater than 37 ft	Spans greater than 37 ft
3	3	40 ft	None	Spans greater than 200 ft	Spans greater than 200 ft
3	4	20 ft	None	Spans greater than 90 ft	Spans greater than 90 ft
3	4	40 ft	None	Spans greater than 50 ft	Spans greater than 50 ft

The ASR methodology results are summarized in Table 26. Combining the results from negative and positive moment, the span restrictions for safe evaluation on platoon and bridge configurations are as follows. The ASR methodology seems less conservative than the LFR, because many configurations that can be safely evaluated under the ASR methodology, seem to have a ratio of less than one under the ASR methodology, and was further investigated. A prominent reason in this drastic change is in the way the resisting moment capacity, that is used in the load rating equation, is calculated. Referring to MBE example A1 for reference, the resisting moment capacity was 1313 kip.ft under the ASR method, and 2914 kip.ft under the LFR method. The ASR rating factor was 0.74, compared to the LFR rating factor of 1.33. Therefore, the results in this study do align with the norm illustrated in the MBE examples.

Table 26: ASR Configuration Summary

ASR					
Configuration			Code not adequate on		
# Spans	# Trucks	Distance	M +	M -	Result
1	2	20 ft	Spans greater than 105 ft	None	Spans greater than 105 ft
1	2	40 ft	Spans greater than 134 ft	None	Spans greater than 134 ft
1	3	20 ft	Spans greater than 105 ft	None	Spans greater than 105 ft
1	3	40 ft	Spans greater than 134 ft	None	Spans greater than 134 ft
1	4	20 ft	Spans greater than 105 ft	None	Spans greater than 105 ft
1	4	40 ft	Spans greater than 134 ft	None	Spans greater than 134 ft
2	2	20 ft	Spans greater than 225 ft	None	Spans greater than 225 ft
2	2	40 ft	Spans greater than 240 ft	None	Spans greater than 240 ft
2	3	20 ft	Spans greater than 180 ft	Spans less than 88 ft	Spans greater than 180 ft and less than 88 ft
2	3	40 ft	Spans greater than 225 ft	Spans less than 120 ft	Spans greater than 225 ft and less than 120 ft
2	4	20 ft	Spans greater than 180 ft	Spans less than 88 ft	Spans greater than 180 ft and less than 88 ft
2	4	40 ft	Spans greater than 225 ft	Spans less than 120 ft	Spans greater than 225 ft and less than 120 ft
3	2	20 ft	None	None	None
3	2	40 ft	None	None	None
3	3	20 ft	Spans greater than 225 ft	Spans less than 70 ft	Spans greater than 225 ft and less than 70 ft
3	3	40 ft	None	Spans less than 80 ft	Spans less than 80 ft
3	4	20 ft	Spans greater than 225 ft	Spans less than 70 ft	Spans greater than 225 ft and less than 70 ft
3	4	40 ft	None	Spans less than 80 ft	Spans less than 80 ft

The LRFR methodology results are examined next and are summarized in Table 27. Combining the results from positive and negative moment, the results are as follows. Two-truck platoons, with a distance of 20 ft and 40 ft, and three-tuck platoons at a distance of 40 ft, on single, two, and three-span configurations, result in an acceptable evaluation under the AASHTO live loads. Areas that require additional attention are three and four truck platoons spaced at a distance of 20 ft, for a single, two, and three-span configuration, and are shown in Table 27.

Table 27: LRFR Configuration Summary

LRFR					
Configuration			Code not adequate on		
# Spans	# Trucks	Distance	M +	M -	Result
1	2	20 ft	None	None	None
1	2	40 ft	None	None	None
1	3	20 ft	Spans greater than 234 ft	None	Spans greater than 234 ft
1	3	40 ft	None	None	None
1	4	20 ft	Spans greater than 186 ft	None	Spans greater than 186 ft
1	4	40 ft	None	None	None
2	2	20 ft	None	None	None
2	2	40 ft	None	None	None
2	3	20 ft	Spans greater than 140 ft	None	Spans greater than 140 ft
2	3	40 ft	None	None	None
2	4	20 ft	Spans greater than 140 ft	None	Spans greater than 140 ft
2	4	40 ft	None	None	None
3	2	20 ft	None	None	None
3	2	40 ft	None	None	None
3	3	20 ft	Spans greater than 180 ft	None	Spans greater than 180 ft
3	3	40 ft	None	None	None
3	4	20 ft	Spans greater than 148 ft	None	Spans greater than 148 ft
3	4	40 ft	None	None	None

Combining the findings from the separate design methodologies are summarized in Table 28. A broad bottom line is that some platoons and bridge configurations, will indeed require some supplemental considerations to what is in the current code, however, there are some platoon configurations under certain bridge characteristics and span lengths that can be safely designed under the current code, without any additional consideration, without the need for alarm.

Load rating evaluation under LFR and LRFR bare similarities. For two-truck platoons at 20 ft and 40 ft distances, and for three truck-platoons at 40 ft, for all span configurations studied, the AASHTO live loads prove to be adequate for evaluations under the LFR and

LRFR. For other configurations, AASHTO live loads are adequate for load rating evaluations, up to a certain span length. However, as previously mentioned in the ASR positive moment evaluation section, the ASR load rating evaluations do not agree as closely with the trend observed frim the LFR and LRFR methodologies.

Table 28: Configuration Summary

Configuration			LFR	ASR	LRFR	Combined
# Spans	# Trucks	Distance	Result	Result	Result	
1	2	20 ft	None	Spans greater than 105 ft	None	Spans greater than 105 ft
1	2	40 ft	None	Spans greater than 134 ft	None	Spans greater than 134 ft
1	3	20 ft	None	Spans greater than 105 ft	Greater than 234 ft	Spans greater than 105 ft
1	3	40 ft	None	Spans greater than 134 ft	None	Spans greater than 134 ft
1	4	20 ft	None	Spans greater than 105 ft	Greater than 186 ft	Spans greater than 105 ft
1	4	40 ft	None	Spans greater than 134 ft	None	Spans greater than 134 ft
2	2	20 ft	None	Spans greater than 225 ft	None	Spans greater than 225 ft
2	2	40 ft	None	Spans greater than 240 ft	None	Spans greater than 240 ft
2	3	20 ft	All Spans	Greater than 180 ft and less than 88 ft	Greater than 140 ft	All spans
2	3	40 ft	Between 130 ft and 161 ft	Greater than 225 ft and less than 120 ft	None	All spans
2	4	20 ft	Between 85 ft and 212 ft	Greater than 180 ft and less than 88 ft	Greater than 140 ft	All spans
2	4	40 ft	All Spans	Greater than 225 ft and less than 120 ft	None	All spans
3	2	20 ft	None	None	None	None
3	2	40 ft	None	None	None	None
3	3	20 ft	Greater than 37 ft	Greater than 225 ft and less than 70 ft	Greater than 180 ft	All spans
3	3	40 ft	Greater than 200 ft	Spans less than 80 ft	None	All spans
3	4	20 ft	Greater than 90 ft	Greater than 225 ft and less than 70 ft	Greater than 148 ft	All spans
3	4	40 ft	Greater than 50 ft	Spans less than 80 ft	None	All spans

With truck platooning technology being sold and implemented in the US, and as bills and legislations are being passed legalizing truck platooning, this study serves as a broad overview on the impact to steel girder bridges. Legislators are not considering the structural integrity of existing bridges, and the need for updating existing codes. It is up to the engineers, to speak up and raise concern. With the lack of evidence that truck platoons, in general, can safely drive over all bridges, and equally, with the lack of evidence that they all pose threat to the safety of the public, this study was merited.

At the conclusion of this study, some bridge configurations have been identified as potentially unsafe to handle particular platoons. With that, it has set the grounds for future researchers to study in greater depth, aspects of the findings. Future direction from the obtained results are plentiful. Some prominent considerations might be as follows. Digging deeper into analyzing the main aspects that play key roles when certain spans length under certain bridge configuration of each design methodology seize to be able to satisfy the current code. Another crucial area is identifying aspects of the current code that would need to be adjusted and reformed for future designs of bridges that are anticipated to welcome platoons. Yet, one more vital area might be developing checks and criterions that enable the engineer to make a decision on whether an existing bridge will be able to safely handle a certain platoon configuration without having to re-run all the design calculations. Those are just some future directions, out of many numerous topics requiring attention. It is important to note that all of this particular study was only developed for steel bridges. Further studies will also need to be developed for reinforced concrete, and prestressed concrete bridges. Therefore, this study is just the beginning, and a lot more needs to be investigated, as we move forward with introducing truck platoons into public roads and bridges.

REFERENCES

AASHTO. 2002. *Standard Specification for Highway Bridges*, 17^{th} Edition, HB-17. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2018. *The Manual for Bridge Evaluation*, 2^{nd} Edition. American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2017. *AASHTO LRFD Bridge Design Specifications*, 8^{th} Edition, LRFDUS-4-M or LRFDSI-4. American Association of State Highway and Transportation Officials, Washington, DC.

Bridge Inspection Manual: Load Ratings. (n.d.). Retrieved from http://onlinemanuals.txdot.gov/txdotmanuals/ins/load_ratings.htm

Bourland, M. C., Chang, B., & Jao, M. (2011*). Verification ff Texas Superheavy Load Criteria for Bridges.*

CalTrans (2008). *Memo To Designers 10-20*. California Department of Transportation.

DeVault, A. (2017). *Two-Truck Platooning. Load Effects of Two-Tuck Platoons on Interstate and Turnpike Bridges in Florida*.

FDOT. (2010, January). *FDOT Modifications to LTS-4*. Retrieved from http://www.fdot.gov/structures/structuresmanual/2010march/Vol8_LRFRmod.pdf

Gao, L. (2013*). Load Rating Highway Bridges in the United States: The State of Practice [Abstract]*. Structural Engineering International.

Kulicki, J. M. (2018, November 20). *Evolution of the AASHTO Bridge Design Specifications*. Lecture presented in University at Buffalo, New York.

Mertz, D. R. (2005). *Load Rating by Load and Resistance Factor Evaluation Method*. National Cooperative Highway Research Program.

Muthu, K., Ibrahim, A., Janardhana, M., & Vijayanand, M. (2017). *Basic structural Analysis* (3rd ed.). New Delhi: I. K. International Publishing House Pvt.

National Conference of State Legislatures. (2018, August 8). *Autonomous Vehicles Self-Driving Vehicles Enacted Legislation*. Retrieved from http://www.ncsl.org/research/transportation/autonomous-vehicles-self-driving-vehicles-enacted-legislation.aspx

NCHRP. 2007. *Legal Truck Loads and AASHTO Legal Loads for Posting*. Transportation Research Board, National Research Council, Washington, DC.

Quickbridge [Computer software]. (n.d.).

STAAD PRO [Computer software]. (n.d.).

TXDOT (2005). *Bridge Design Manual – LRFD.* Texas Department of Transportation.

TXDOT (2015). *Preferred Practices for Steel Bridge Design, Fabrication, and Erection.* – Texas Department of Transportation.

U.S. Department of Transportation. (2018, June). *Bridge Formula Weights.* Retrieved from https://ops.fhwa.dot.gov/freight/publications/brdg_frm_wghts/index.htm

Wood, S. M. (2007). *Long-Term Effects of Super Heavy-Weight Vehicles on Bridges.*

Yarnold, M. T., Weidner, J. S., (2018). *Truck Platoon Impacts on Steel Girder Bridges*. Journal of Bridge Engineering.

(2018, February 5). *Platooning Trucks to Cut Cost and Improve Efficiency.* Retrieved from https://www.energy.gov/eere/articles/platooning-trucks-cut-cost-and-improve-efficiency

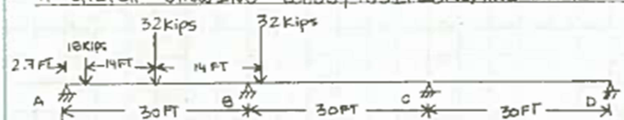
(2017). *What is truck platooning*? Retrieved from https://www.acea.be/uploads/publications/Platooning_roadmap.pdf

APPENDIX A

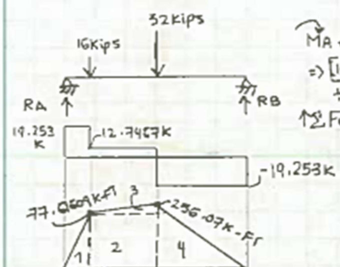
HAND CALCULATION

MOMENT CALCULATION OF A THREE SPAN	CHECK FOR THE EXCEL SPREADSHEET	RITA TOHME SEPTEMBER 8^{TH}, 2018

A- SKETCH SHOWING LOADS, POSITIONS, AND DIMENSIONS



B- CALCULATIONS



$\overset{\curvearrowright}{M_A} = 0$

$\Rightarrow [(16K)(2.7') + (32K)(16.7')] \div 30' = R_B = 19.253$ kips

$\uparrow \Sigma F_y = 0 \Rightarrow R_A = 28.746$ kips

: shear diagram

: moment diagram

$A_1 = \frac{1}{2} \times 2.7' \times 77.61609$ K-FT $= 104.78$, $\vec{x}_1 = 2/3 \times 2.7' = 1.8'$

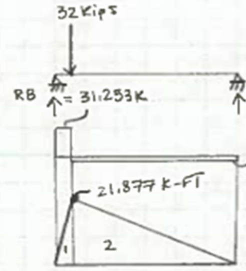
$A_2 = 77.61609$ K-FT $\times$ 14FT $= 1086.625$, $\vec{x}_2 = 2.7' + \frac{14'}{2} = 9.7'$

$A_3 = \frac{1}{2} \times 14' \times (256.07 - 77.61609)$ K-FT $= 1249.2$, $\vec{x}_3 = 2.7 + \frac{2 \times 14'}{3} = 12.03'$

$A_4 = \frac{1}{2} \times 13.3' \times 256.07$ K-FT $= 1702.87$, $\vec{x}_4 = 16.7 + \frac{13.3}{3} = 21.13'$

$\vec{x} = \frac{\Sigma A_i \vec{x_i}}{\Sigma A_i} = 14.9$ FT ∴ $\overleftarrow{x} = 15.1$ FT

$\Sigma A_i = 4143.45$ FT^2



$\overset{\curvearrowright}{M_B} = 0$

$\Rightarrow 32K \times 0.7FT / 30FT = R_C$

$R_C = 0.746$ kips

$\uparrow \Sigma F = 0 \Rightarrow R_B = 31.253K$

$A_1 = \frac{1}{2} \times 21.877$ K-FT $\times 0.7$FT $= 7.657$, $\vec{x}_1 = \frac{2}{3} \times 0.7 = 0.467$ FT

$A_2 = \frac{1}{2} \times 21.877 \times 29.3$FT $= 320.5$, $\vec{x}_2 = 0.7 + \frac{29.3}{3} = 10.46$FT

$\vec{x} = \frac{\Sigma A_i \vec{x_i}}{\Sigma A_i} = 10.234$ FT, $\overleftarrow{x} = 19.766$ FT

$\Sigma A_i = 328.157$

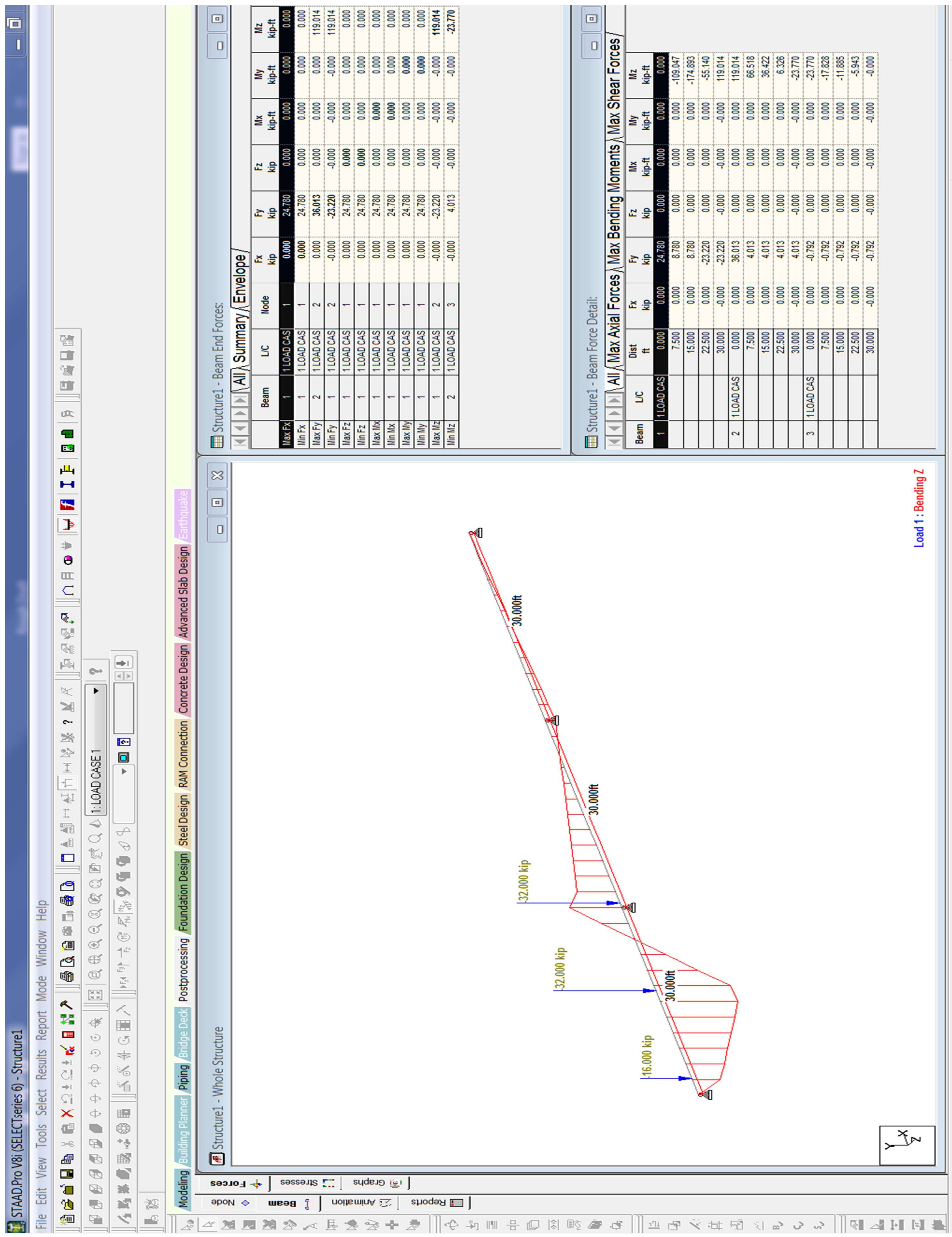
$$\therefore \begin{cases} 30\cancel{M_A}^{0} + 2M_B(30+30) + 30M_C & -6(4143.45 \times 14.9/30 + 328.157 \times 19.766/30) \\ 30M_B + 2M_C(30+30) + \cancel{M_D}^{0} & = -6(328.157 \times 10.234/30) \end{cases}$$

$$\begin{cases} 120M_B + 30M_C = -13644.75 \\ 30M_B + 120M_C = -671.67 \end{cases}$$

$$\therefore \left. \begin{array}{l} M_B = -119.796 \text{ Kip-FT} \\ M_C = +24.35 \text{ kip-FT} \end{array} \right) \text{MATCH} \checkmark \text{ OKAY}$$

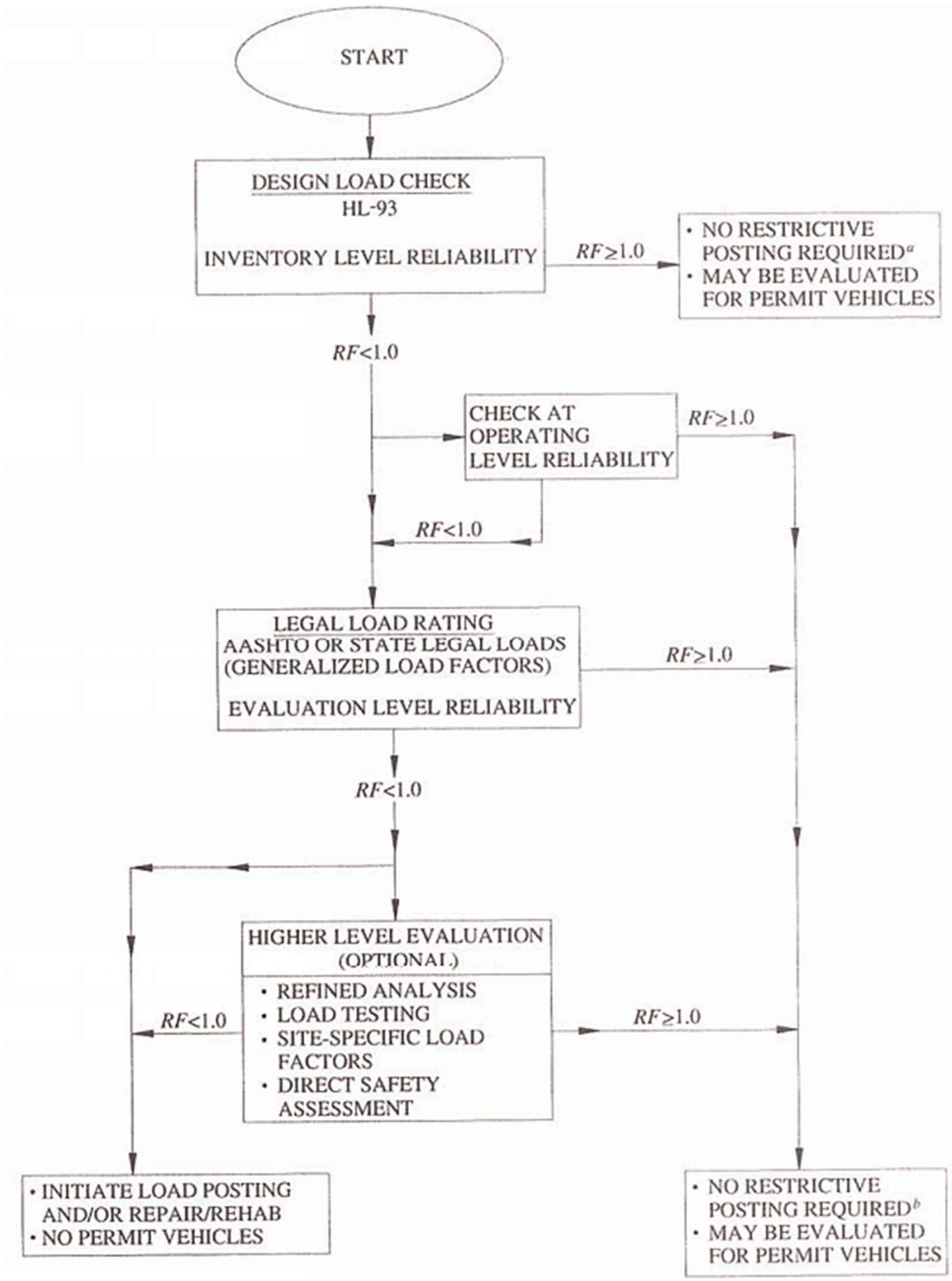
APPENDIX B

STTAD RESULTS



APPENDIX C

LOAD RATING FLOW CHART (MBE)



[a] For routinely permitted on highways of various states under grandfather exclusions to federal weight laws.

[b] For legal loads that comply with federal weight limits and Formula B.

APPENDIX D

MBE TABLE 6.5.2.1-1

		DATE BUILT-STEEL UNKNOWN								
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel	$1^1/_8$ in. and under	Over $1^1/_8$ in. to 2 in. incl.
AASHTO Designation[a]						M 94 (1961)	M 95 (1961)	M 96 (1961)		
ASTM Designation[a]						A7 (1967)	A94 (1966)	A8 (1961)	A94	A94
Minimum Tensile Strength	F_u	52,000	60,000			60,000	70,000	90,000	75,000	72,000
Minimum Yield Point	F_y	26,000	30,000	33,000	36,000	33,000	45,000	55,000	50,000	47,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
	$0.46F_y$	NOT APPLICABLE								
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending. (use whichever is smaller)	Gross[*] Section $0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
• When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	Net Section $0.50F_u$	26,000	30,000	30,000	30,000	30,000	35,000	45,000	37,000	36,000
	Net Section $0.46F_u$	NOT APPLICABLE								
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]	$0.55F_y$	14,000	16,000	18,000	20,000	18,000	24,000	30,000	27,000	25,000

$$F_b = \frac{91\times10^6 C_{1x}}{(F.S.)S_{xc}}\left(\frac{1_{yc}}{1}\right)\sqrt{0.772\frac{J}{1_{yc}}+9.87\left(\frac{d}{1}\right)^2} \le 0.55F_y$$

C_b = $1.75 + 1.05\ (M_1/M_2) + 0.3\ (M_1/M_2)2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$F.S.$ = Factor of Safety at Inventory Level = 1.82

		DATE BUILT-STEEL UNKNOWN								
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel	$1^1/_8$ in. and under	Over $1^1/_8$ in. to 2 in. incl.
Compression in concentrically loaded columns[c] with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		148.4	138.1	131.7	126.1	131.7	112.8	102.0	107.0	110.4
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \le C_c$		$12{,}260 - 0.28\left(\frac{KL}{r}\right)^2$	$14{,}150 - 0.37\left(\frac{KL}{r}\right)^2$	$15{,}570 - 0.45\left(\frac{KL}{r}\right)^2$	$16{,}980 - 0.53\left(\frac{KL}{r}\right)^2$	$15{,}570 - 0.45\left(\frac{KL}{r}\right)^2$	$21{,}230 - 0.83\left(\frac{KL}{r}\right)^2$	$25{,}940 - 1.25\left(\frac{KL}{r}\right)^2$	$23{,}580 - 1.03\left(\frac{KL}{r}\right)^2$	$22{,}170 - 0.91\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{135{,}008{,}740}{\left(\frac{KL}{r}\right)^2}$ when $\frac{KL}{r} \ge C_c$ with $F.S. = 2.12$										
Shear in girder webs, gross section		8,500	9,500	11,000	12,000	11,000	14,000	17,500	16,500	15,500
Bearing on milled stiffeners and other steel parts in contact Stress in extreme fiber of pins	$0.80F_y$	20,000	24,000	26,000	29,000	26,000	36,000	44,000	40,000	37,000
Bearing on pins not subject to rotation		20,000	24,000	26,000	29,000	26,000	32,000	40,000	40,000	37,000
Bearing on pins subject to rotation (such as rockers and hinges)		10,000	12,000	13,000	14,000	13,000	16,000	18,000	20,000	18,000
Shear in pins	$0.40F_y$	10,000	12,000	13,000	14,000	13,000	18,000	22,000	20,000	18,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	70,000	81,000			81,000	94,500	121,000	100,000	97,500

[a] Number in parentheses represents the last year these specifications were printed.

[b] For the use of larger C_b values, see *Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures*, Third Edition, p. 135. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

ℓ = length of unsupported flange between lateral connections, knee braces, or other points of support, in.

ℓ_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in.^{4}

d = depth of girder, in.

J = $\frac{\left[(bt^3)_c + (bt^3)_t + Dt_w^3\right]}{3}$ where b and t represent the flange width and thickness of the compression and tension flange, D is the web depth, and t_w is the web thickness, in.^{4}

S_{xc} = Section modulus with respect to the compression flange, in.^{3}

[c] E = modulus of elasticity of steel

r = governing radius of gyration

L = actual unbraced length

K = effective length factor

Note: The formulas do not apply to members with variable moment of inertia.

		$1^1/_2$ in. max	$^1/_2$ in. max	Over $2^1/_2$ in. to 4 in. incl.	$^3/_4$ in. and under	To $2^1/_2$ in. incl. (A 514) All thick (A 517)	Over 4 in. to 5 in. incl/ (A 588) Over $^3/_4$ in. to $1^1/_2$ in. incl.
AASHTO Designation[a]							
ASTM Designation[a]		A572	A572	A514	A242, A440, A441	A514/A517	A242, A440, A441, A588
Minimum Tensile Strength	F_u	60,000	80,000		70,000	115,000	67,000
Minimum Yield Point	F_y	45,000	65,000	90,000	50,000	100,000	46,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.55F_y$	25,000	36,000	N.A.	27,000	55,000	25,000
	$0.46F_y$	NOT APPLICABLE		48,300	53,000	53,000	N.A.
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending (use whichever is smaller)	Gross* Section $0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000
• When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	Net Section $0.50F_u$	30,000	40,000	N.A.	35,000	N.A.	33,500
	Net Section $0.46F_u$	NOT APPLICABLE		48,300	N.A.	53,000	N.A.
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]	$0.55F_y$	25,000	36,000	49,000	27,000	55,000	25,000

$$F_b = \frac{91 \times 10^6 C_b}{(F.S.)\, S_{xc}} \left(\frac{I_{yc}}{l}\right) \sqrt{0.772 \frac{J}{I_{yc}} + 9.87 \left(\frac{d}{l}\right)^2} \leq 0.55 F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$F.S.$ = Factor of Safety at Inventory Level = 1.82

		$1^{1}/_{2}$ in. max	$^{1}/_{2}$ in. max	Over $2^{1}/_{2}$ in. to 4 in. incl.	$^{3}/_{4}$ in. and under	To $2^{1}/_{2}$ in. incl. (A 514) All thick (A 517)	Over 4 in. to 5 in. incl. (A 588) Over $^{3}/_{4}$ in. to $1^{1}/_{2}$ in. incl.
Compression in concentrically loaded columns[c] with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		112.8	93.8	79.8	107.0	75.7	111.6
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \le C_c$		$21{,}230 - 0.83\left(\frac{KL}{r}\right)^2$	$30{,}660 - 1.74\left(\frac{KL}{r}\right)^2$	$42{,}450 - 3.34\left(\frac{KL}{r}\right)^2$	$23{,}580 - 1.03\left(\frac{KL}{r}\right)^2$	$47{,}170 - 4.12\left(\frac{KL}{r}\right)^2$	$21{,}700 - 0.87\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{135{,}008{,}740}{\left(\frac{KL}{r}\right)^2}$ when $KL \ge C_c$ with $F.S. = 2.12$							
Shear in girder webs, gross section		15,000	22,000	30,000	17,000	30,000	15,000
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.80F_y$	37,000	52,000	72,000	40,000	80,000	37,000
Bearing on pins not subject to rotation		37,000	52,000	72,000	40,000	80,000	37,000
Bearing on pins subject to rotation (such as rockers and hinges)		18,000	26,000	36,000	20,000	40,000	18,000
Shear in pins	$0.40F_y$	18,000	26,000	36,000	20,000	40,000	18,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	81,000	108,000	142,000	94,500	155,000	90,500

Table 6B.5.2.1-1—Inventory Rating Allowable Stresses, psi (continued)

		$1^{1}/_{2}$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1^{1}/_{2}$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
AASHTO Designation[a]					M 188 (1983)
ASTM Designation[a]		A572	A572	A242, A440, A441, A588, A572	A441 (1985)
Minimum Tensile Strength	F_u	70,000	75,000	63,000	60,000
Minimum Yield Point	F_y	55,000	60,000	42,000	40,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^{1}/_{4}$-in. diameter, such as perforations.	$0.55F_y$	30,000	33,000	23,000	22,000
	0.46	NOT APPLICABLE			
Axial tension in members with holes for high strength	Gross* Section	30,000	33,000	23,000	22,000

bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending • When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	use whichever is smaller	$0.55F_y$ Net Section $0.50F_u$ Net Section $0.46\ F_u$	35,000	37,500	31,500	30,000
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.		$0.55F_y$	30,000	33,000	23,000	22,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]		$0.55F_y$	30,000	33,000	23,000	

$$F_b = \frac{91 x 10^6 C_b}{(F.S.)\, S_{xc}} \left(\frac{I_{yc}}{l}\right) \sqrt{0.772 \frac{J}{I_{yc}} + 9.87 \left(\frac{d}{l}\right)^2} \leq 0.55F_y$$

C_b = $1.75 + 1.05\ (M_1/M_2) + 0.3\ (M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$F.S.$ = Factor of Safety at Inventory Level = 1.82

		$1^{1}/_{2}$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1^{1}/_{2}$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
Compression in concentrically loaded columns[c]					
with $C_c = \sqrt{\dfrac{2\pi^2 E}{F_y}}$		102.0	97.7	116.7	
$F_a = \dfrac{F_y}{F.S.}\left[1 - \dfrac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\dfrac{KL}{r} \le C_c$		$25{,}940 - 1.25\left(\frac{KL}{r}\right)^2$	$28{,}300 - 1.48\left(\frac{KL}{r}\right)^2$	$19{,}810 - 0.73\left(\frac{KL}{r}\right)^2$	
$F_a = \dfrac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \dfrac{135{,}008{,}740}{\left(\frac{KL}{r}\right)^2}$ when $KL \ge C_c$ with $F.S. = 2.12$					
Shear in girder webs, gross section		18,000	20,000	14,000	
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.80F_y$	44,000	48,000	34,000	32,000
Bearing on pins not subject to rotation		44,000	48,000	34,000	32,000
Bearing on pins subject to rotation (such as rockers and hinges)		22,000	24,000	17,000	16,000
Shear in pins	$0.40F_y$	22,000	24,000	17,000	
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.35F_u$	94,500	101,000	85,000	81,000

APPENDIX E

MBE TABLE 6.5.2.1-2

		DATE BUILT-STEEL UNKNOWN						
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel
AASHTO Designation[a]						M 94 (1961)	M 95 (1961)	M 96 (1961)
ASTM Designation[a]						A7 (1967)	A4 (1966)	A8 (1961)
Minimum Tensile Strength	F_u	52,000	60,000			60,000	70,000	90,000
Minimum Yield Point	F_y	26,000	30,000	33,000	36,000	33,000	45,000	55,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.	$0.75F_y$	19,500	22,500	24,500	27,000	24,500		
	$0.60F_u$	NOT APPLICABLE					33,500	41,000
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending • When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.	use whichever is smaller Gross[*] Section $0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000
	Net Section $0.67F_u$	35,000	40,000	40,000	40,000	40,000	46,500	60,000
	Net Section $0.60F_u$	NOT APPLICABLE						
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.	$0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000
Compression in extreme fibers of rolled shapes, girders, and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]	$0.75F_y$	19,500	22,500	24,500	27,000	24,500	33,500	41,000

$$F_b = \frac{91 x 10^6 C_b}{(F.S.) S_{xc}}\left(\frac{1_{yc}}{1}\right)\sqrt{0.772\frac{J}{1_{yc}}+9.87\left(\frac{d}{1}\right)^2} \leq 0.75F_y$$

C_b = $0.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \leq 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams: M_1/M_1 is positive when the moments cause reverse curvature and negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Inventory Level = 1.34

		DATE BUILT-STEEL UNKNOWN						
		Prior to 1905	1905 to 1936	1936 to 1963	After 1963	Carbon Steel	Silicon Steel Over 2 in. to 4 in. incl.	Nickel Steel
Compression in concentrically loaded columns[c]								
with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$ when $\frac{KL}{r} \le C_c$		148.4	138.1	131.7	126.1	131.7	112.8	102.0
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \ge C_c$		$15{,}290 - 0.35\left(\frac{KL}{r}\right)^2$	$17{,}650 - 0.46\left(\frac{KL}{r}\right)^2$	$19{,}410 - 0.56\left(\frac{KL}{r}\right)^2$	$21{,}180 - 0.67\left(\frac{KL}{r}\right)^2$	$19{,}410 - 0.56\left(\frac{KL}{r}\right)^2$	$26{,}470 - 1.04\left(\frac{KL}{r}\right)^2$	$32{,}350 - 1.55\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{168{,}363{,}840}{\left(\frac{KL}{r}\right)^2}$ with $F.S. = 1.70$								
Shear in girder webs, gross section	$0.45F_y$	11,500	13,500	15,000	16,000	15,000	20,000	24,500
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	23,000	27,000	29,500	32,000	29,500	40,500	49,500
Bearing on pins not subject to rotation	$0.90F_y$	23,000	27,000	29,500	32,000	29,500	40,500	49,500
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	14,000	16,500	18,000	19,500	18,000	24,500	30,000
Shear in pins	$0.55F_y$	14,000	16,500	18,000	19,500	18,000	24,500	30,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	96,000	111,000	111,000	111,000	111,000	129,500	166,500

[a] Number in parentheses represents the last year these specifications were printed.

[b] For the use of larger C_b values, see Structural Stability Research Council Guide to Stability Design Criteria for Metal Structures, Third Edition, p. 135. If cover plates are used, the allowable static stress at the point of theoretical cutoff shall be as determined by the formula.

ℓ = length of unsupported flange between lateral connections, knee braces, or other points of support, in.

ℓ_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web, in.^{4}

d = depth of girder, in.

J = $\frac{\left[(bt^3)_c + (bt^3)_t + Dt_w^3\right]}{3}$, in.^{4}, where b and t represent the flange width and thickness of the compression and tension flange, D is the web depth, and t_w is the web thickness.

S_{xc} = Section modulus with respect to the compression flange, in.^{3}

[c] E = modulus of elasticity of steel
r = governing radius of gyration
L = actual unbraced length
K = effective length factor

Note: The formulae do not apply to members with variable moment of inertia.

			8 in. and Under	$1^{1}/_{8}$ in. and Under	Over $1^{1}/_{8}$ in. to 2 in. incl.	$1^{1}/_{2}$ in. max	$^{1}/_{2}$ in. max	Over $2^{1}/_{2}$ in. to 4 in. incl.	$^{3}/_{4}$ in. and under 4 in. and under (A 588)
AASHTO Designation[a]									
ASTM Designation[a]			A36	A94	A94	A572	A572	A514	A242, A440, A441, A588, A572
Minimum Tensile Strength		F_u	58,000	75,000	72,000	60,000	80,000	105,000	70,000
Minimum Yield Point		F_y	36,000	50,000	47,000	45,000	65,000	90,000	50,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^{1}/_{4}$-in. diameter, such as perforations.		$0.75F_y$	27,000	37,500	35,000	33,500	48,500	N.A.	37,500
		$0.60F_u$		NOT APPLICABLE				63,000	N.A.
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending	use whichever is smaller	Gross Section $0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500
• When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^{1}/_{4}$-in. diameter, such as perforations, shall be deducted.		Net Section $0.67F_u$	38,000	50,000	48,000	40,000	53,000	N.A.	46,500
		Net Section $0.60F_u$			NOT APPLICABLE			63,000	N.A.
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.		$0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]		$0.75F_y$	27,000	37,500	35,000	33,500	48,500	67,500	37,500

$$F_b = \frac{91 x 10^6 C_b}{(F.S.) S_{xc}} \left(\frac{1_{yc}}{1}\right) \sqrt{0.772 \frac{J}{1_{yc}} + 9.87 \left(\frac{d}{1}\right)^2} \le 0.75 F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

= 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

F.S. = Factor of Safety at Operating Level = 1.34

		8 in. and under	$1^1/_8$ in. and under	Over $1^1/_8$ in. to 2 in. incl.	$1^1/_2$ in. max	$^1/_2$ in. max	Over $2^1/_2$ in. to 4 in. incl.	$^3/_4$ in. and under 4 in. and under (A 588)
Compression in concentrically loaded columns[c]								
with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$ when $\frac{KL}{r} \le C_c$		126.1	107.0	110.4	112.8	93.8	79.8	107.0
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \ge C_c$		$21{,}180 - 0.67\left(\frac{KL}{r}\right)^2$	$29{,}410 - 1.28\left(\frac{KL}{r}\right)^2$	$27{,}650 - 1.13\left(\frac{KL}{r}\right)^2$	$26{,}470 - 1.04\left(\frac{KL}{r}\right)^2$	$38{,}240 - 2.17\left(\frac{KL}{r}\right)^2$	$52{,}940 - 4.16\left(\frac{KL}{r}\right)^2$	$29{,}410 - 1.28\left(\frac{KL}{r}\right)^2$
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{168{,}363{,}840}{\left(\frac{KL}{r}\right)^2}$ with $F.S. = 1.70$								
Shear in girder webs, gross section	$0.45F_y$	16,000	22,500	21,000	20,000	29,000	40,500	22,500
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	32,000	45,000	42,000	40,500	58,500	81,000	45,000
Bearing on pins not subject to rotation	$0.90F_y$	32,000	45,000	42,000	40,500	58,500	81,000	45,000
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	19,500	27,500	25,500	24,500	35,500	49,500	27,500
Shear in pins	$0.55F_y$	19,500	27,500	25,500	24,500	35,500	49,500	27,500
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	107,000	138,500	133,000	171,000	148,000	194,000	129,500

			To $2^1/_2$ in. incl (A 514) All thick (A 517)	Over 4 in. to 5 in. incl. (A 588) Over $^3/_4$ in. to $1^1/_2$ in. incl.	$1^1/_2$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1^1/_2$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
AASHTO Designation[a]								
ASTM Designation[a]			A514–A517	A242, A440, A441, A588	A572	A572	A242, A440, A441, A588, A572	A441
Minimum Tensile Strength		F_u	115,000	67,000	70,000	75,000	63,000	60,000
Minimum Yield Point		F_y	100,000	46,000	55,000	60,000	42,000	40,000
Axial tension in members with no holes for high-strength bolts or rivets. Use net section when member has any open holes larger than $1^1/_4$-in. diameter, such as perforations.		$0.75F_y$ $0.60F_u$	75,000 69,000	34,500	41,000	45,000 NOT APPLICABLE		30,000
Axial tension in members with holes for high-strength bolts or rivets and tension in extreme fiber of rolled shapes, girders, and built-up sections subject to bending.	use whichever is smaller	Gross[c] Section $0.75F_y$	75,000	34,500	41,000	45,000	31,500	30,000
• When the area of holes deducted for high-strength bolts or rivets is more than 15 percent of the gross area, that area in excess of 15 percent shall be deducted from the gross area in determining stress on the gross section. In determining gross section, any open holes larger than $1^1/_4$-in. diameter, such as perforations, shall be deducted.		Net Section $0.67F_u$	N.A.	44,500	46,500	50,000	42,000	40,000
		Net Section $0.60F_u$	69,000			NOT APPLICABLE		
Axial tension in members without holes. Axial compression, gross section: stiffeners of plate girders. Compression in splice material, gross section.		$0.75F_y$	75,000	34,500	41,000	45,000	31,500	30,000
Compression in extreme fibers of rolled shapes, girders and built-up sections, subject to bending, gross section, when compression flange is: (A) Supported laterally its full length by embedment in concrete (B) Partially supported or unsupported[b]		$0.75F_y$	75,000	34,500	41,000	45,000	31,500	

$$F_b = \frac{91\times10^6 C_b}{(F.S.)S_{xc}}\left(\frac{1_{yc}}{1}\right)\sqrt{0.772\frac{J}{1_{yc}}+9.87\left(\frac{d}{1}\right)^2} \le 0.75F_y$$

C_b = $1.75 + 1.05\,(M_1/M_2) + 0.3\,(M_1/M_2)^2 \le 2.3$ where M_1 is the smaller and M_2 is the larger end moment in the unbraced segment of the beams; M_1/M_2 is positive when the moments cause reverse curvature and negative when bent in single curvature.

C_b = 1.0 for unbraced cantilevers and for members where the moment within a significant portion of the unbraced segment is greater than or equal to the larger of the segment end moments.

$F.S.$ = Factor of Safety at Inventory Level = 1.34

		To $2\frac{1}{2}$ in. incl. (A 511) All thick (A 517)	Over 4in. to 5in. incl. (A 588) Over $\frac{3}{4}$ in. to $1\frac{1}{2}$ in. incl.	$1\frac{1}{2}$ in. max	1 in. max	Over 5 in. to 8 in. incl. (A 588) Over $1\frac{1}{2}$ in. to 4 in. incl.	Over 4 in. to 8 in. incl.
Compression in concentrically loaded columns[c] with $C_c = \sqrt{\frac{2\pi^2 E}{F_y}}$		75.7	111.6	102.0	97.7	116.7	
when $\frac{KL}{r} \le C_c$							
$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{KL}{r}\right)^2 F_y}{4\pi^2 E}\right]$ when $\frac{KL}{r} \ge C_c$		$58{,}820 - 5.14\left(\frac{KL}{r}\right)^2$	$27{,}060 - 1.09\left(\frac{KL}{r}\right)^2$	$32{,}350 - 1.55\left(\frac{KL}{r}\right)^2$	$35{,}290 - 1.85\left(\frac{KL}{r}\right)^2$	$24{,}710 - 0.91\left(\frac{KL}{r}\right)^2$	
$F_a = \frac{\pi^2 E}{F.S.\left(\frac{KL}{r}\right)^2} = \frac{168{,}363{,}840}{\left(\frac{KL}{r}\right)^2}$ with $F.S. = 1.70$							
Shear in girder webs, gross section	$0.45F_y$	45,000	20,500	24,500	27,000	18,500	18,000
Bearing on milled stiffeners and other steel parts in contact. Stress in extreme fiber of pins	$0.90F_y$	90,000	41,000	49,500	54,000	37,500	36,000
Bearing on pins not subject to rotation	$0.90F_y$	90,000	41,000	49,500	54,000	37,500	36,000
Bearing on pins subject to rotation (such as rockers and hinges)	$0.55F_y$	55,000	25,000	30,000	33,000	23,000	22,000
Shear in pins	$0.55F_y$	55,000	25,000	30,000	33,000	23,000	22,000
Bearing on Power-Driven Rivets and high-strength bolts (or as limited by allowable bearing on the Fasteners)	$1.85F_u$	213,000	124,000	129,500	138,500	116,500	111,000

APPENDIX F

MBE APPENDIX L6B

L6B.1—GENERAL

When using the Load Factor Method, the capacity C in the basic load rating Eq. 6B.4.1-1 is based on procedures in the latest edition of AASHTO's *Standard Specifications for Highway Bridges* (AASHTO Standard Specifications). This Appendix summarizes the capacity determination for typical bridge members of steel, reinforced concrete, or prestressed concrete. For more conditions not covered in this Appendix, the AASHTO Standard Specifications should be used.

The formulas shown below have been taken from the AASHTO Standard Specifications. All equation and article numbers cited below refer to this Specification, except for a few article numbers that refer to Appendix L6B itself. The notation used in the formulas is as defined in the AASHTO Standard Specifications.

L6B.2—CAPACITY OF STEEL MEMBERS (PART D, STRENGTH DESIGN METHOD)

L6B.2.1—Sections in Bending

The capacities specified in L6B.2.1.1 and L6B.2.1.2 are applicable to compact rolled or welded beams and girders, satisfying the applicable cross-sectional limitations, which are rolled or fabricated from steels with a specified minimum yield strength between 33,000 and 50,000 psi. The capacities specified in L6B.2.1.3 through L6B.2.1.5 are applicable to noncompact rolled, riveted, or welded beams and girders satisfying the applicable cross-sectional limitations, which are rolled or fabricated from steels with a minimum specified yield strength between 33,000 and 100,000 psi. The equations found in L6B.2.1.1 through L6B.2.1.5 are not applicable to hybrid girders.

L6B.2.1.1—Compact, Braced, Noncomposite

$$C = F_y Z \qquad (10\text{-}92)$$

L6B.2.1.2—Compact, Composite

Positive Moment Sections

For composite positive moment sections satisfying the cross-sectional limitations specified in Article 10.50.1.1.2:

In simple spans or in continuous spans with compact noncomposite negative-moment pier sections:

$$C = M_n$$

where M_n is determined according to Eq. 10-129b or Eq. 10-129c, as applicable, in Article 10.50.1.1.2. For steel with F_y = 33,000 psi, $\beta = 0.9$ in Article 10.50.1.1.2.

In continuous spans with noncompact noncomposite or composite negative-moment pier sections:

Tension and Compression Flange

$$C = F_y$$

Alternatively, C may be taken as M_n, where M_n is determined according to Eq. 10-129d in Article 10.50.1.1.2.

According to the preceding requirements, the capacity of a composite positive moment section satisfying the cross-sectional limitations for a compact section specified in Article 10.50.1.1.2 will be at or just below the full plastic moment capacity, M_p, in simple spans and in continuous spans with compact pier sections. In this case, the dead and live load moments are to be used in the basic load-rating equation to compute a rating factor for the section. In continuous spans with noncompact pier sections, the capacity of a compact composite positive moment section will typically be taken equal to the yield stress, F_y. In this case, the dead and live load stresses in each flange are to be used in the basic load rating equation to compute a rating factor for each flange. In either case, however, the web slenderness requirement for the positive moment section given by Eq. 10-129 is to be checked using the depth of the web in compression at the plastic moment, D_{cp}. The elastic depth of the web in compression, D_c, is not to be used in checking the web slenderness requirement for these sections.

Negative Moment Sections

For composite negative moment sections satisfying the cross-sectional limitations specified in Article 10.50.2.1:

$$C = M_u$$

where M_u is determined according to the provisions of Article 10.50.2.1.

L6B.2.1.3—Noncompact, Noncomposite

The lesser of:

$$C = F_y S_{xt} \tag{10-98}$$

or if Eq. 10-101 is satisfied:

$$C = F_y S_{xc} \tag{10-99}$$

where:

$$F_{cr} = \left(4,400\frac{t}{b}\right)^2 \le F_y$$

R_b shall be calculated from the provisions of Article 10.48.4.1 with F_{cr} substituted for the term M_r/S_{xc} when Eq. 10-103b applies.

If Eq. 10-101 is not satisfied:

$$C = F_{cr} S_{xc} Rb \le M_u$$

where M_u is determined according to the provisions of Article 10.48.4.1.

L6B.2.1.4—Noncompact, Composite, Positive Moment Section

Tension Flange

$$C = F_y$$

Compression Flange

$$C = F_y R_b$$

When R_b is determined from Eq. 10-103b, F_y shall be substituted for the term M_r/S_{xc} and A_{fc} shall be taken as the effective combined transformed area of the top flange and concrete deck that yields D_c calculated in accordance with Article 10.50(b). The resulting R_b factor shall be distributed to the top flange and concrete deck in proportion to their relative stiffness.

Since D_c is a function of the dead-to-live load stress ratio according to the provisions of Article 10.50(b), an iterative procedure may be required to determine the rating factor for the compression flange.

L6B.2.1.5—Noncompact, Composite, Negative Moment Section

Tension Flange

$C = F_y$

Compression Flange

If Eq. 10-101 is satisfied:

$C = F_{cr}R_b$

where:

$$F_{cr} = \left(4,400\frac{t}{b}\right)^2 \leq F_y$$

R_b shall be calculated from the provisions of Article 10.48.4.1 with F_{cr} substituted for the term M_r/S_{xc} when Eq. 10-103b applies.

If Eq. 10-101 is not satisfied:

$C = F_{cr}R_b \leq M_u / S_{xc}$

where M_u and S_{xc} are determined according to the provisions of Article 10.48.4.1.

D_c of the composite section consisting of the steel section plus the longitudinal reinforcement may conservatively be used in lieu of D_c calculated according to the provisions of Article 10.50(b).

L6B.2.2—Sections in Shear

$C = V_u$ (10-113 or 10-114)

where V_u is found in accordance with Article 10.48.8.1.

APPENDIX G

BRIDGE WEIGHT FORMULA RESULTS

Axel Wt.	Axel distance (ft)	Number of trucks
10	0.0	1 truck
20	10.0	
20	4.2	
15	17.7	
15	4.2	
10	20 40	2 trucks
20	10.0	
20	4.2	
15	17.7	
15	4.2	
10	20 40	3 trucks
20	10.0	
20	4.2	
15	17.7	
15	4.2	
10	20 40	4 trucks
20	10.0	
20	4.2	
15	17.7	
15	4.2	

L	N	Wt. Allow (lb)	Wt. Actual (lb)	Actual/ Allow	# of trucks	truck Dist. (ft)
10.0	2	40000	30000	0.75	1	
4.2	2	34167	40000	1.17	1	
17.7	2	47667	35000	0.73	1	
4.2	2	34167	30000	0.88	1	
20.0	2	50000	25000	0.50	2	20
40.0	2	70000	25000	0.36	2	40
14.2	3	57250	50000	0.87	1	
21.8	3	68750	55000	0.80	1	
21.8	3	68750	50000	0.73	1	
24.2	3	72250	40000	0.55	2	20
44.2	3	102250	40000	0.39	2	40
30.0	3	81000	45000	0.56	2	20
50.0	3	111000	45000	0.41	2	40
14.2	3	57250	50000	0.87	1	
31.8	4	105667	65000	0.62	1	
26.0	4	94000	70000	0.74	1	
41.8	4	125667	60000	0.48	2	20
61.8	4	165667	60000	0.36	2	40
34.2	4	110333	60000	0.54	2	20
54.2	4	150333	60000	0.40	2	40
34.2	4	110333	65000	0.59	2	20
54.2	4	150333	65000	0.43	2	40
31.8	4	105667	65000	0.62	1	
36.0	5	138000	80000	0.58	1	
46.0	5	163000	80000	0.49	2	20
66.0	5	213000	80000	0.38	2	40
51.8	5	177583	80000	0.45	2	20
71.8	5	227583	80000	0.35	2	40
38.3	5	143833	80000	0.56	2	20
58.3	5	193833	80000	0.41	2	40
51.8	5	177583	80000	0.45	2	20
71.8	5	227583	80000	0.35	2	40
36.0	5	138000	80000	0.58	1	
56.0	6	222000	90000	0.41	2	20

L	N	Wt. Allow (lb)	Wt. Actual (lb)	Actual/ Allow	# of trucks	truck Dist. (ft)
76.0	6	282000	90000	0.32	2	40
56.0	6	222000	100000	0.45	2	20
76.0	6	282000	100000	0.35	2	40
76.0	6	282000	100000	0.35	2	20
96.0	6	342000	100000	0.29	2	40
56.0	6	222000	100000	0.45	2	20
76.0	6	282000	100000	0.35	2	40
56.0	6	222000	95000	0.43	2	20
76.0	6	282000	95000	0.34	2	40
56.00	6	222000	90000	0.41	2	20
76.00	6	282000	90000	0.32	2	40
66.00	7	291000	110000	0.38	2	20
86.00	7	361000	110000	0.30	2	40
86.00	7	361000	110000	0.30	2	20
106.00	7	431000	110000	0.26	2	40
73.67	7	317833	115000	0.36	2	20
93.67	7	387833	115000	0.30	2	40
60.17	7	270583	110000	0.41	2	20
80.17	7	340583	110000	0.32	2	40
76.00	7	326000	105000	0.32	3	20
116.00	7	466000	105000	0.23	3	40
66.0	7	291000	110000	0.38	2	20
86.0	7	361000	110000	0.30	2	40
70.2	8	346667	130000	0.38	2	20
90.2	8	426667	130000	0.30	2	40
77.8	8	377333	135000	0.36	2	20
97.8	8	457333	135000	0.30	2	40
77.8	8	377333	130000	0.34	2	20
97.8	8	457333	130000	0.28	2	40
80.2	8	386667	120000	0.31	3	20
120.2	8	546667	120000	0.22	3	40
86.0	8	410000	125000	0.30	3	20
126.0	8	570000	125000	0.22	3	40
70	8	346667	130000	0.38	2	20
90	8	426667	130000	0.30	2	40
88	9	467250	145000	0.31	2	20
108	9	557250	145000	0.26	2	40
82	9	441000	150000	0.34	2	20
102	9	531000	150000	0.28	2	40
97.8	9	512250	140000	0.27	3	20
138	9	692250	140000	0.20	3	40
90.2	9	477750	140000	0.29	3	20
130	9	657750	140000	0.21	3	40
90	9	477750	145000	0.30	3	20
130	9	657750	145000	0.22	3	40
87.8	9	467250	145000	0.31	3	20
128	9	647250	145000	0.22	3	40
87.8	9	467250	145000	0.31	2	20
92.0	10	538000	160000	0.30	2	20
102.0	9	531000	160000	0.30	3	20
107.8	9	557250	160000	0.29	3	20
94.3	9	496500	160000	0.32	3	20
107.8	9	557250	160000	0.29	2	20
92.0	9	486000	160000	0.33	2	20
102.0	9	531000	160000	0.30	3	20
107.8	9	557250	160000	0.29	3	20
94.3	9	496500	160000	0.32	3	20
107.8	9	557250	160000	0.29	2	20

L	N	Wt. Allow (lb)	Wt. Actual (lb)	Actual/ Allow	# of trucks	2 truck Dist. (ft)
92.0	9	486000	160000	0.33	2	20
107.8	9	557250	145000	0.26	2	40
112.0	10	638000	160000	0.25	2	40
142.0	9	711000	160000	0.23	3	40
147.8	9	737250	160000	0.22	3	40
134.3	9	676500	160000	0.24	3	40
147.8	9	737250	160000	0.22	2	40
112.0	9	576000	160000	0.28	2	40
142.0	9	711000	160000	0.23	3	40
147.8	9	737250	160000	0.22	3	40
134.3	9	676500	160000	0.24	3	40
127.8	9	647250	160000	0.25	2	40
132.0	9	666000	160000	0.24	2	40
87.8	9	467250	145000	0.31	2	20
92.0	10	538000	160000	0.30	2	20
102.0	10	588000	160000	0.27	3	20
107.8	10	617167	160000	0.26	3	20
94.3	10	549667	160000	0.29	3	20
107.8	10	617167	160000	0.26	3	20
92.0	10	538000	160000	0.30	2	20
102.0	10	588000	160000	0.27	3	20
107.8	10	617167	160000	0.26	3	20
94.3	10	549667	160000	0.29	3	20
107.8	10	617167	160000	0.26	3	20
92.0	10	538000	160000	0.30	2	20
107.8	10	617167	145000	0.23	2	40
112.0	10	638000	160000	0.25	2	40
142.0	10	788000	160000	0.20	3	40
147.8	10	817167	160000	0.20	3	40
134.3	10	749667	160000	0.21	3	40
147.8	10	817167	160000	0.20	3	40
112.0	10	638000	160000	0.25	2	40
142.0	10	788000	160000	0.20	3	40
147.8	10	817167	160000	0.20	3	40
134.3	10	749667	160000	0.21	3	40
147.8	10	817167	160000	0.20	3	40
112.0	10	638000	160000	0.25	2	40
92.0	10	538000	160000	0.30	2	20
112.0	11	700000	170000	0.24	3	20
112.0	11	700000	180000	0.26	3	20
112.0	11	700000	180000	0.26	3	20
112.0	11	700000	175000	0.25	3	20
112.0	11	700000	175000	0.25	2	20
112.0	11	700000	160000	0.23	2	40
152.0	11	920000	170000	0.18	3	40
152.0	11	920000	180000	0.20	3	40
152.0	11	920000	180000	0.20	3	40
152.0	11	920000	175000	0.19	3	40
152.0	11	920000	175000	0.19	2	40
112.0	11	700000	170000	0.24	3	20
122.0	12	822000	190000	0.23	3	20
116.2	12	787000	200000	0.25	3	20
129.7	12	868000	195000	0.22	3	20
116.2	12	787000	190000	0.24	3	20
132.0	12	882000	185000	0.21	3	20
122.0	12	822000	190000	0.23	3	20
116.2	12	787000	200000	0.25	3	20
129.7	12	868000	195000	0.22	3	20

L	N	Wt. Allow (lb)	Wt. Actual (lb)	Actual/ Allow	# of trucks	2 truck Dist. (ft)
116.2	12	787000	190000	0.24	3	20
152.0	11	920000	170000	0.18	3	40
162.0	12	1062000	190000	0.18	3	40
156.2	12	1027000	200000	0.19	3	40
169.7	12	1108000	195000	0.18	3	40
156.2	12	1027000	190000	0.19	3	40
192.0	12	1242000	185000	0.15	3	40
162.0	12	1062000	190000	0.18	3	40
156.2	12	1027000	200000	0.19	3	40
169.7	12	1108000	195000	0.18	3	40
156.2	12	1027000	190000	0.19	3	40
122.0	12	822000	190000	0.23	3	20
126.2	13	916083	210000	0.23	3	20
133.8	13	965917	215000	0.22	3	20
133.8	13	965917	210000	0.22	3	20
136.2	13	981083	200000	0.20	4	20
142.0	13	1019000	205000	0.20	3	20
126.2	13	916083	210000	0.23	3	20
133.8	13	965917	215000	0.22	3	20
133.8	13	965917	210000	0.22	3	20
162.0	12	1062000	190000	0.18	3	40
166.2	13	1176083	210000	0.18	3	40
173.8	13	1225917	215000	0.18	3	40
173.8	13	1225917	210000	0.17	4	40
196.2	13	1371083	200000	0.15	3	40
202.0	13	1409000	205000	0.15	3	40
166.2	13	1176083	210000	0.18	3	40
173.8	13	1225917	215000	0.18	3	40
173.8	13	1225917	210000	0.17	3	40
126.2	13	916083	210000	0.23	3	20
143.8	14	1108833	225000	0.20	3	20
138.0	14	1068000	230000	0.22	3	20
153.8	14	1178833	220000	0.19	4	20
146.2	14	1125167	220000	0.20	4	20
146.2	14	1125167	225000	0.20	3	20
143.8	14	1108833	225000	0.20	3	20
138.0	14	1068000	230000	0.22	3	20
166.2	13	1176083	210000	0.18	3	40
183.8	14	1388833	225000	0.16	3	40
178.0	14	1348000	230000	0.17	3	40
213.8	14	1598833	220000	0.14	4	40
206.2	14	1545167	220000	0.14	4	40
206.2	14	1545167	225000	0.15	3	40
183.8	14	1388833	225000	0.16	3	40
178.0	14	1348000	230000	0.17	3	40
143.8	14	1108833	225000	0.20	3	20
148.0	15	1218000	240000	0.20	3	20
158.0	15	1293000	240000	0.19	4	20
163.8	15	1336750	240000	0.18	4	20
150.3	15	1235500	240000	0.19	4	20
163.8	15	1336750	240000	0.18	3	20
148.0	15	1218000	240000	0.20	3	20
183.8	14	1388833	225000	0.16	3	40
188.0	15	1518000	240000	0.16	3	40
218.0	15	1743000	240000	0.14	4	40
223.8	15	1786750	240000	0.13	4	40
210.3	15	1685500	240000	0.14	4	40
223.8	15	1786750	240000	0.13	3	40
188.0	15	1518000	240000	0.16	3	40

APPENDIX H

CALTRANS DECK SLAB THICKNESS

MEMO TO DESIGNERS 10-20 • MAY 2008

ATTACHMENT 2

Table 10-20.1(a) Deck Slab Thickness and Reinforcement Schedule

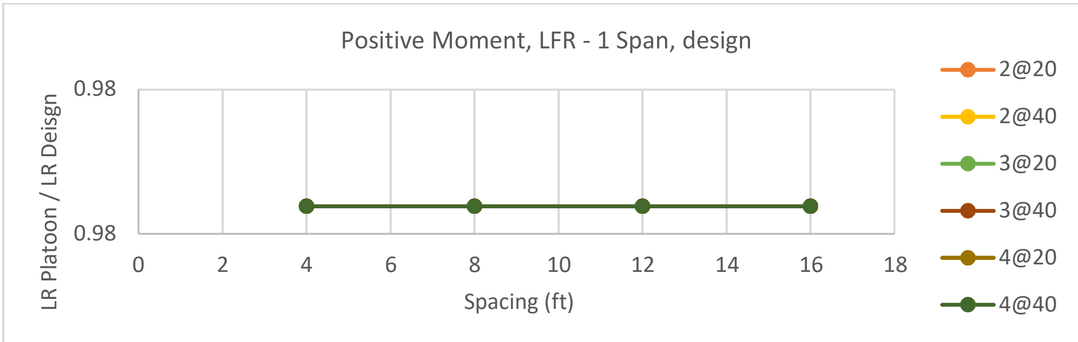
REINFORCED CONCRETE BOX & STEEL GIRDERS w/ flange width >12"and < 24"						
"S"	"t"	Dimension	Transverse Bars		"D" Bars	"G" Bars
Girder CL to CL Spacing	Top Slab Thickness	"F"	Size	Spacing [1]	#5 Bars	#4 Bars
4'- 0"	7"	6"	#5	12"	3	2
4'- 3"	7"	6"	#5	12"	3	2
4'- 6"	7"	6"	#5	12"	3	2
4'- 9"	7"	7"	#5	12"	3	2
5'- 0"	7"	7"	#5	12"	4	2
5'- 3"	7"	7"	#5	12"	4	3
5'- 6"	7"	8"	#5	12"	4	3
5'- 9"	7"	8"	#5	11"	4	3
6'- 0"	7 1/8"	9"	#5	11"	5	3
6'- 3"	7 1/8"	9"	#5	11"	5	3
6'- 6"	7 1/4"	9"	#5	11"	5	3
6'- 9"	7 3/8"	10"	#5	11"	5	3
7'- 0"	7 1/2"	10"	#5	10"	6	3
7'- 3"	7 1/2"	11"	#5	10"	6	3
7'- 6"	7 5/8"	11"	#5	10"	6	3
7'- 9"	7 3/4"	11"	#5	10"	6	3
8'- 0"	7 3/4"	1'- 0"	#5	10"	7	3
8'- 3"	7 7/8"	1'- 0"	#5	10"	7	4
8'- 6"	8"	1'- 1"	#5	10"	7	4
8'- 9"	8 1/8"	1'- 1"	#5	10"	7	4
9'- 0"	8 1/8"	1'- 1"	#5	10"	7	4
9'- 3"	8 1/4"	1'- 2"	#5	10"	8	4
9'- 6"	8 3/8"	1'- 2"	#5	10"	8	4
9'- 9"	8 3/8"	1'- 2"	#5	10"	8	4
10'- 0"	8 1/2"	1'- 3"	#6	12"	10	4
10'- 3"	8 5/8"	1'- 3"	#6	11"	11	4
10'- 6"	8 5/8"	1'- 4"	#6	11"	11	4
10'- 9"	8 3/4"	1'- 4"	#6	11"	11	4
11'- 0"	8 7/8"	1'- 4"	#6	11"	11	4
11'- 3"	8 7/8"	1'- 5"	#6	11"	12	5
11'- 6"	9"	1'- 5"	#6	11"	12	5
11'- 9"	9 1/8"	1'- 6"	#6	11"	12	5
12'- 0"	9 1/8"	1'- 6"	#6	10"	13	5
12'- 3"	9 1/4"	1'- 6"	#6	10"	13	5
12'- 6"	9 3/8"	1'- 7"	#6	10"	13	5
12'- 9"	9 1/2"	1'- 7"	#6	10"	14	5
13'- 0"	9 1/2"	1'- 7"	#6	10"	14	5
13'- 3"	9 5/8"	1'- 8"	#6	10"	14	5
13'- 6"	9 3/4"	1'- 8"	#6	10"	14	5
13'- 9"	9 3/4"	1'- 9"	#6	10"	14	5
14'- 0"	9 7/8"	1'- 9"	#6	10"	14	5
14'- 3"	10"	1'- 9"	#6	10"	14	5
14'- 6"	10 1/8"	1'- 10"	#6	10"	15	5
14'- 9"	10 1/4"	1'- 10"	#6	10"	15	5
15'- 0"	10 3/8"	1'- 11"	#6	10"	15	5

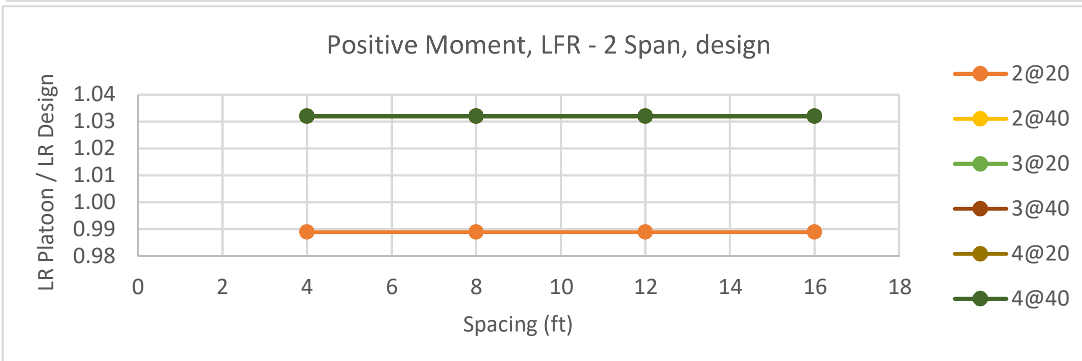
Note: The minimum distance from centerline girder to the negative moment design section has been assumed to be 3".

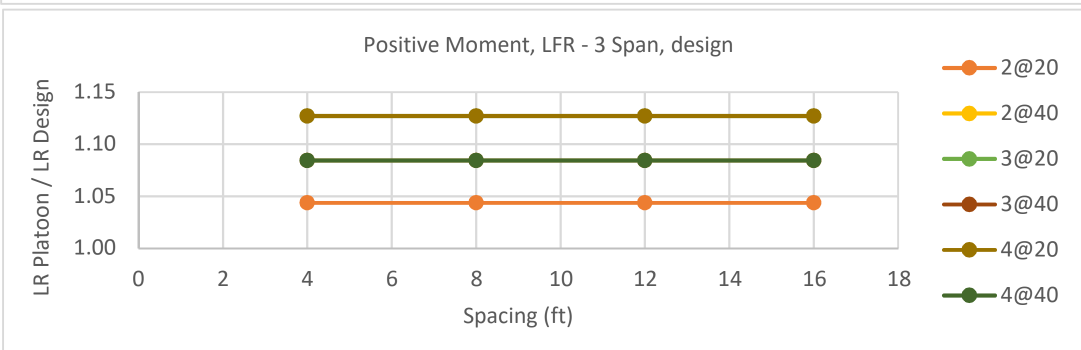
1. See *Standard Plans BO-5* for additional information on transverse deck reinforcement

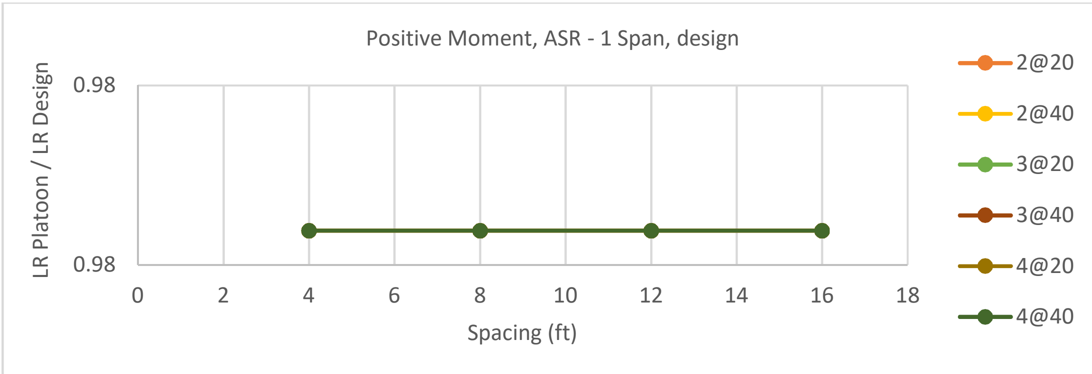
APPENDIX I

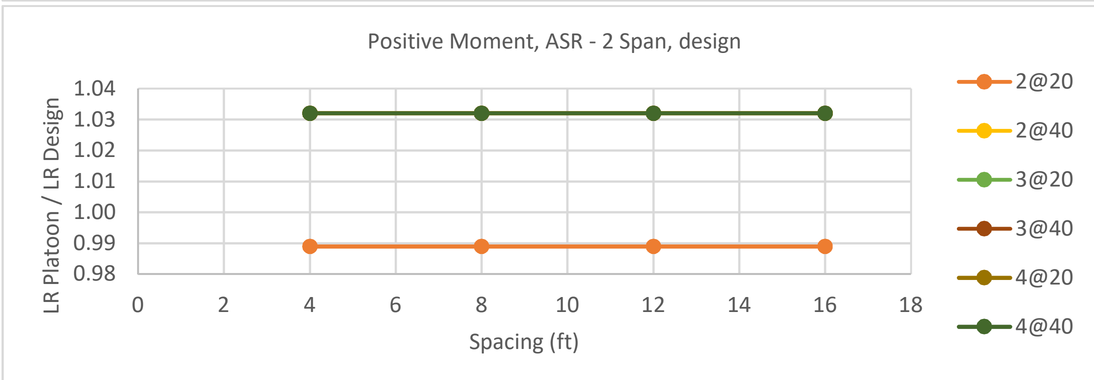
GIRDER SPACING VARIES RESULTS

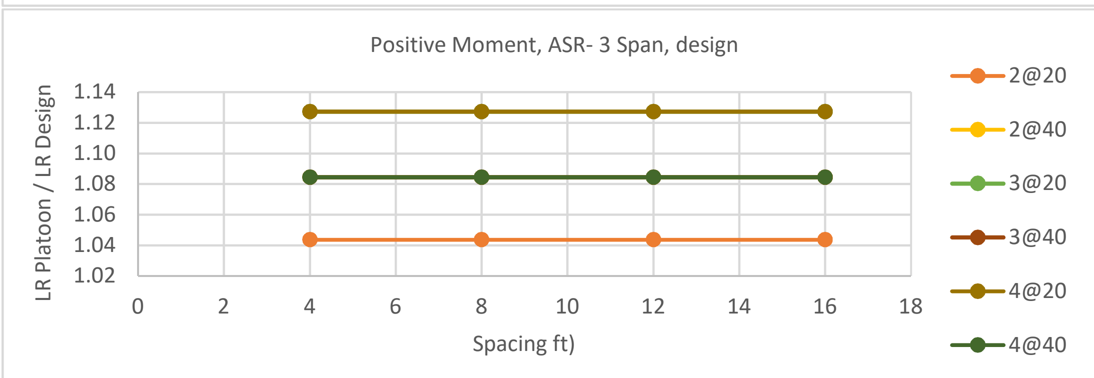


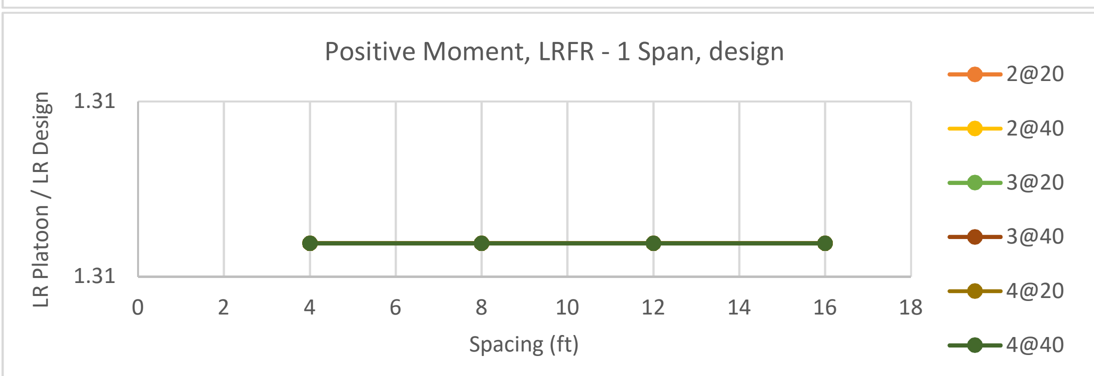


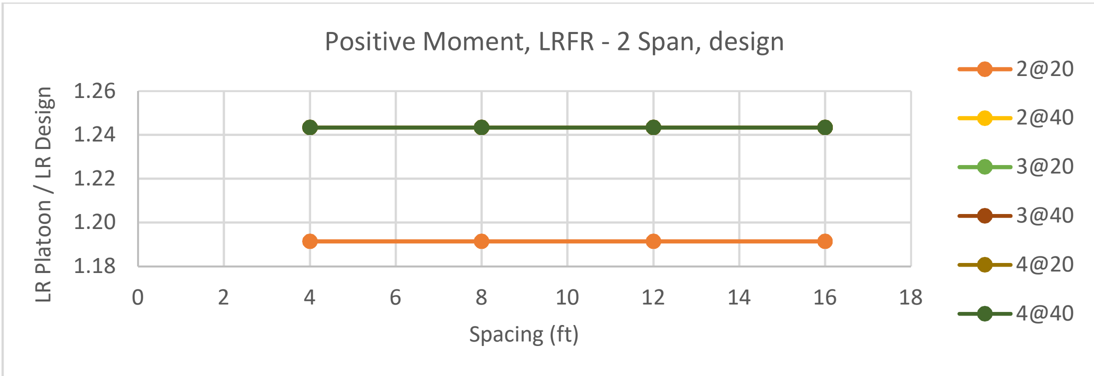


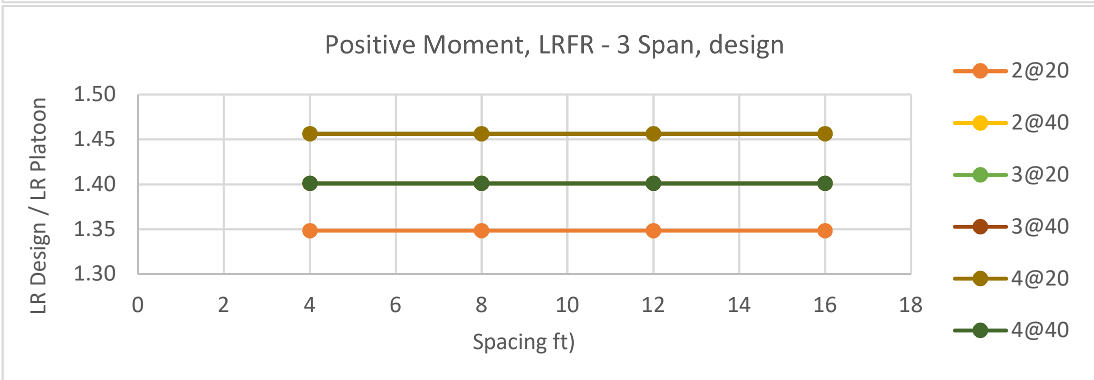

Positive Moment, ASR - 1 Span, design
LR Platoon / LR Design
0.98
0.98
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

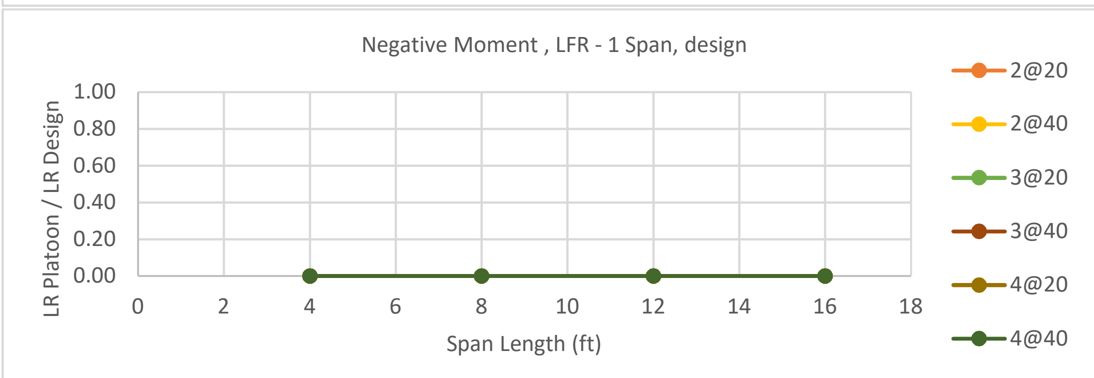

Positive Moment, ASR - 2 Span, design
LR Platoon / LR Design
1.04
1.03
1.02
1.01
1.00
0.99
0.98
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

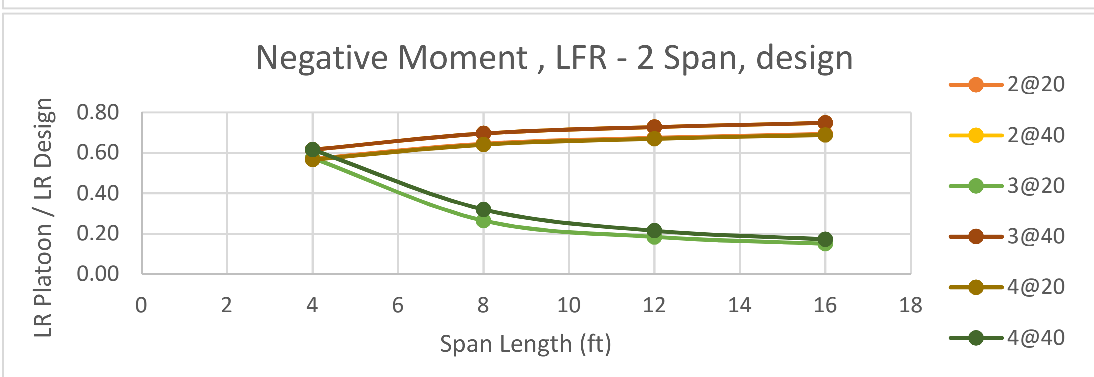

Positive Moment, ASR- 3 Span, design
LR Platoon / LR Design
1.14
1.12
1.10
1.08
1.06
1.04
1.02
0
2
4
6
8
10
12
14
16
18
Spacing ft)
2@20
2@40
3@20
3@40
4@20
4@40

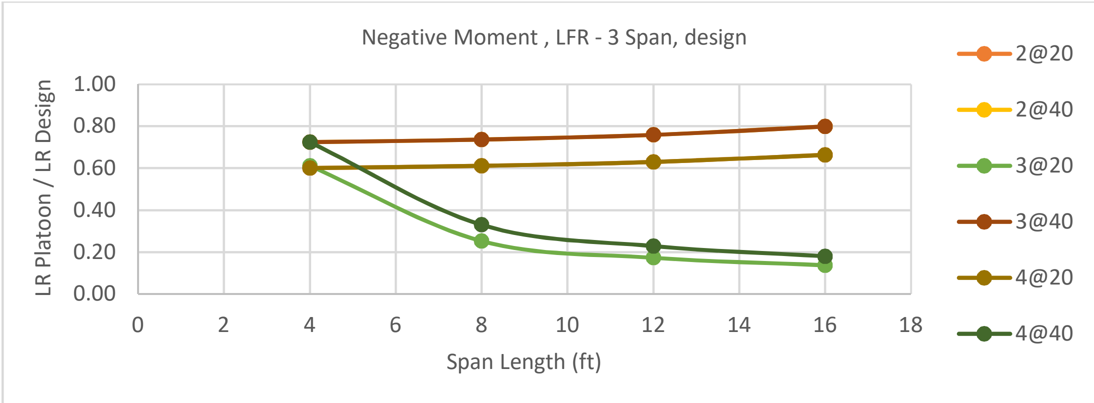

Positive Moment, LRFR - 1 Span, design
LR Platoon / LR Design
1.31
1.31
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

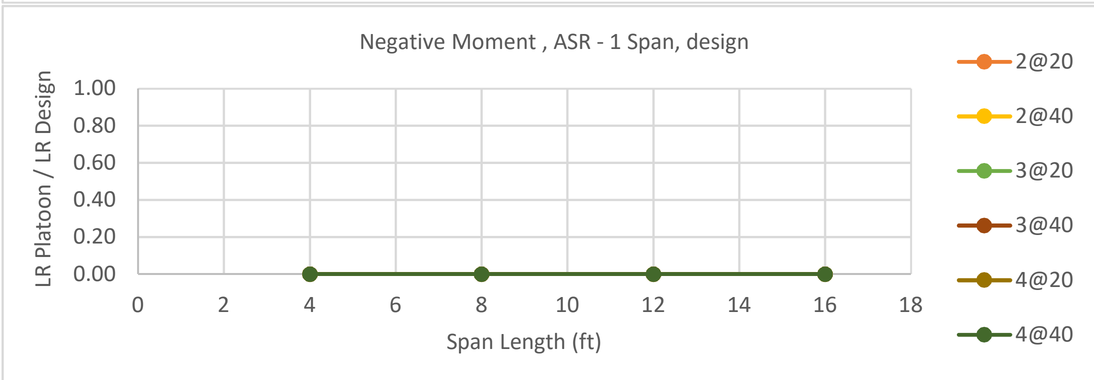

Positive Moment, LRFR - 2 Span, design
LR Platoon / LR Design
1.26
1.24
1.22
1.20
1.18
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

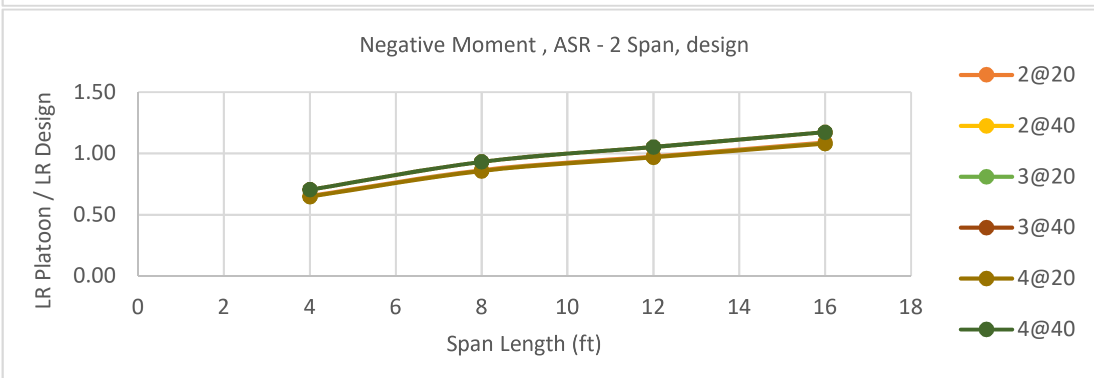

Positive Moment, LRFR - 3 Span, design
LR Design / LR Platoon
1.50
1.45
1.40
1.35
1.30
0
2
4
6
8
10
12
14
16
18
Spacing ft)
2@20
2@40
3@20
3@40
4@20
4@40

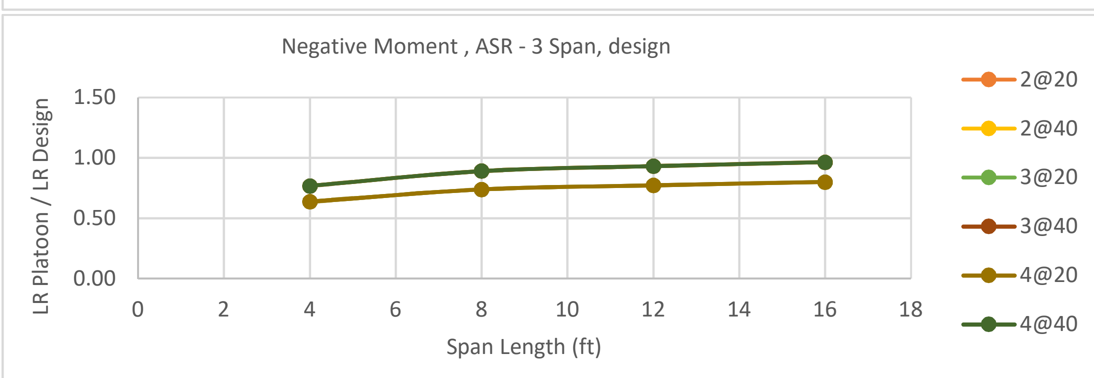

Negative Moment , LFR - 1 Span, design
LR Platoon / LR Design
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

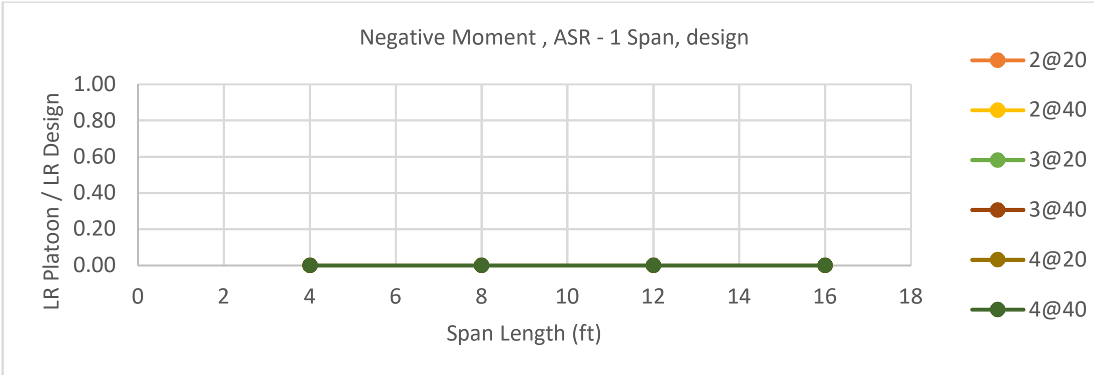

Negative Moment , LFR - 2 Span, design
LR Platoon / LR Design
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

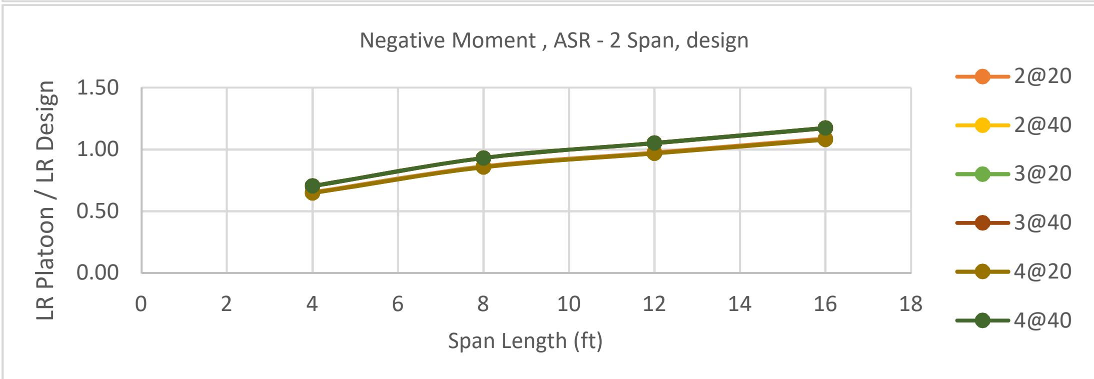

Negative Moment , LFR - 3 Span, design
LR Platoon / LR Design
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

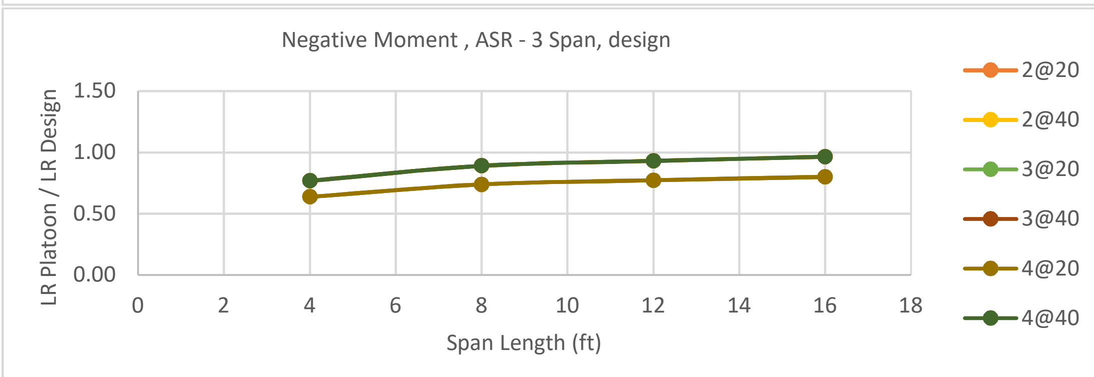

Negative Moment , ASR - 1 Span, design
LR Platoon / LR Design
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

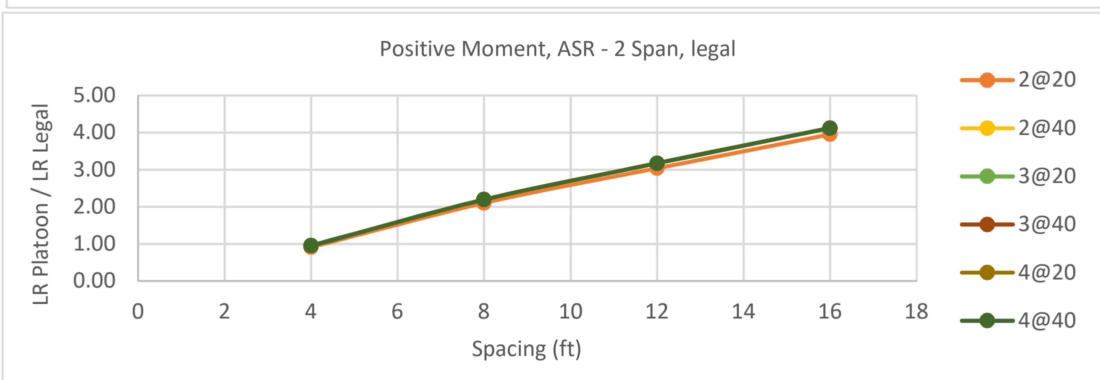

Negative Moment , ASR - 2 Span, design
LR Platoon / LR Design
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

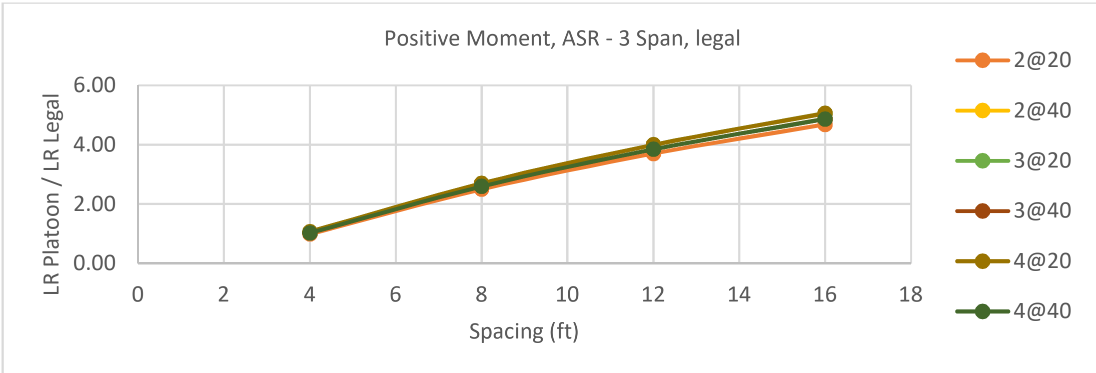

Negative Moment , ASR - 3 Span, design
LR Platoon / LR Design
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

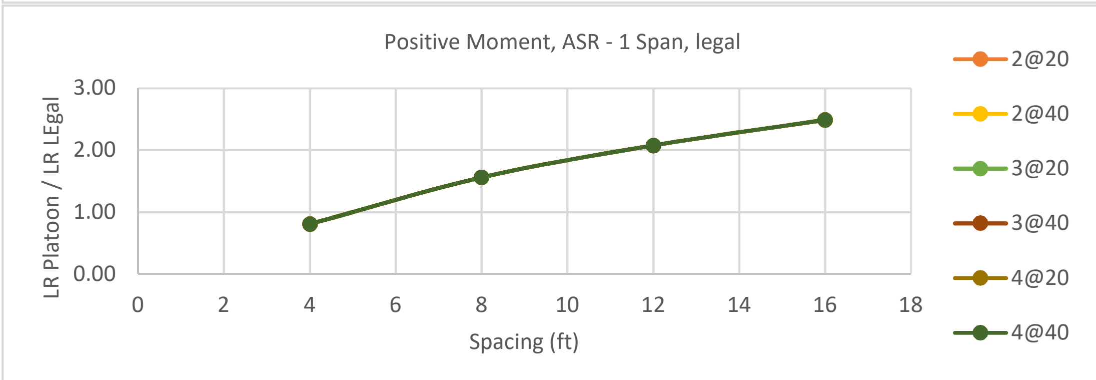

Negative Moment , ASR - 1 Span, design
LR Platoon / LR Design
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

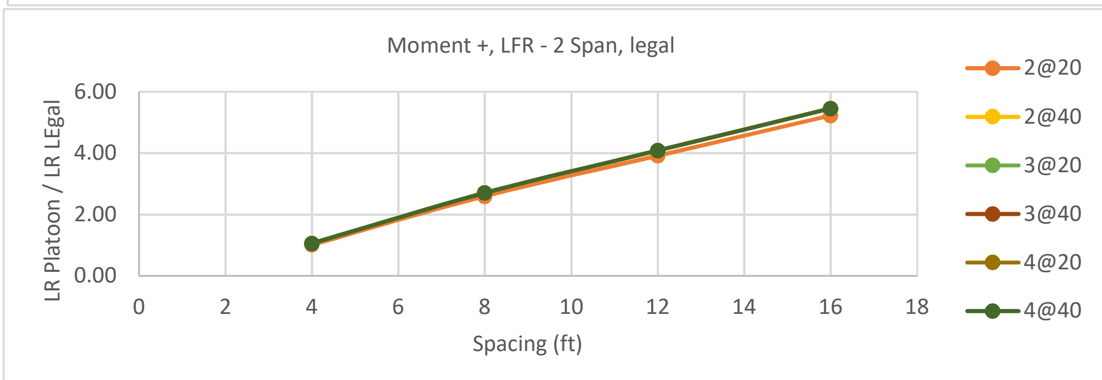

Negative Moment , ASR - 2 Span, design
LR Platoon / LR Design
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40

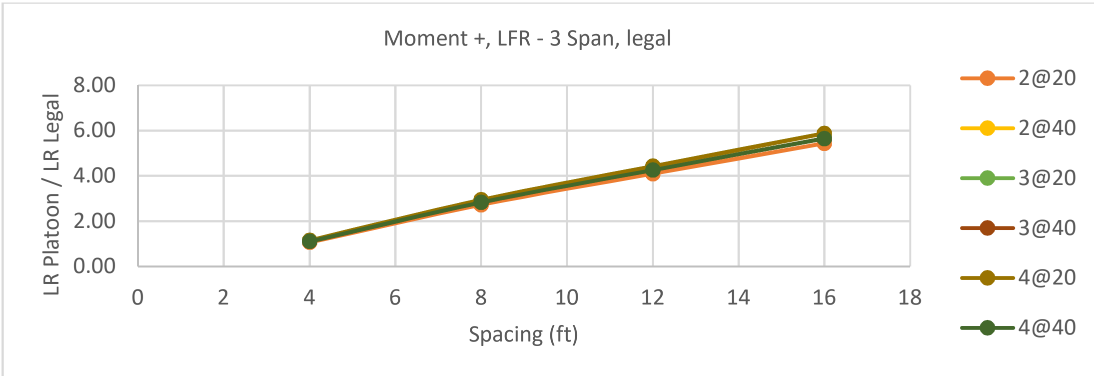

Negative Moment , ASR - 3 Span, design
LR Platoon / LR Design
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Span Length (ft)
2@20
2@40
3@20
3@40
4@20
4@40


Positive Moment, ASR - 2 Span, legal
LR Platoon / LR Legal
5.00
4.00
3.00
2.00
1.00
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

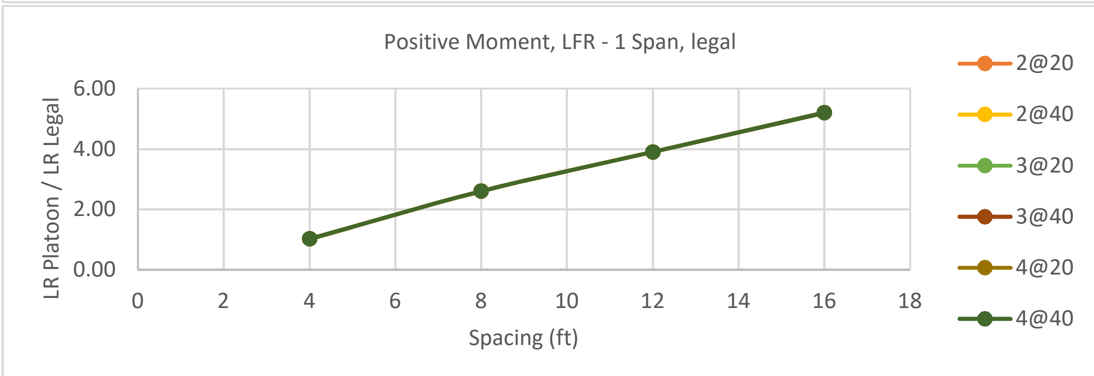

Positive Moment, ASR - 3 Span, legal
LR Platoon / LR Legal
6.00
4.00
2.00
0.00
0 2 4 6 8 10 12 14 16 18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40


Positive Moment, ASR - 1 Span, legal
LR Platoon / LR LEgal
3.00
2.00
1.00
0.00
0 2 4 6 8 10 12 14 16 18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

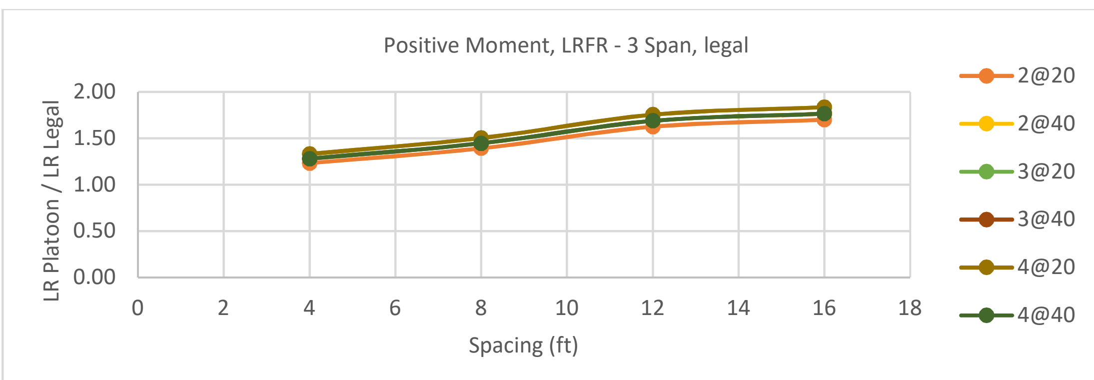

Moment +, LFR - 2 Span, legal
LR Platoon / LR LEgal
6.00
4.00
2.00
0.00
0 2 4 6 8 10 12 14 16 18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40



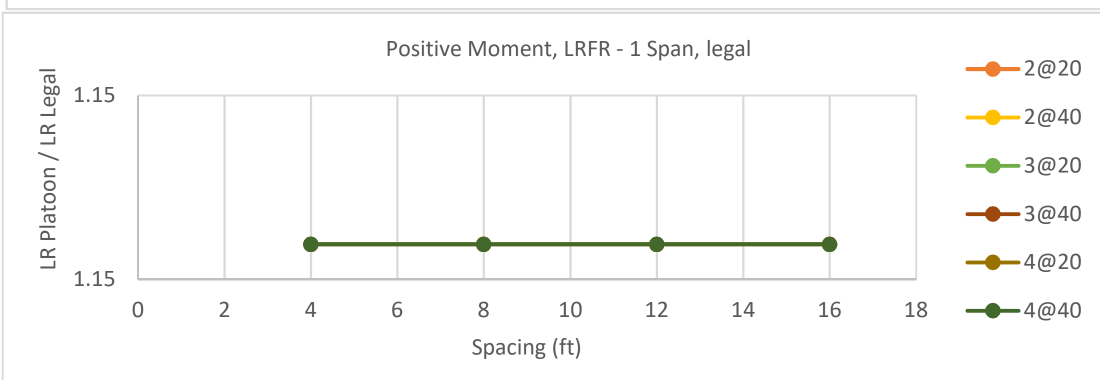
Moment +, LFR - 3 Span, legal
LR Platoon / LR Legal
8.00
6.00
4.00
2.00
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40



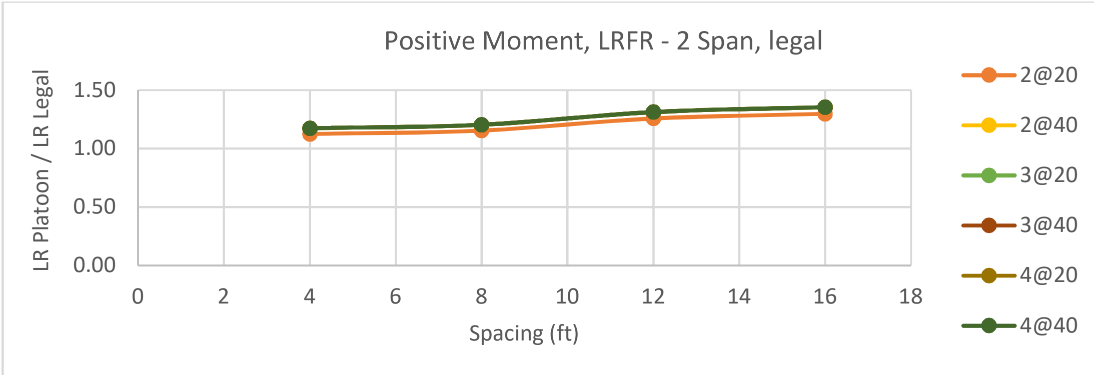
Positive Moment, LFR - 1 Span, legal
LR Platoon / LR Legal
6.00
4.00
2.00
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

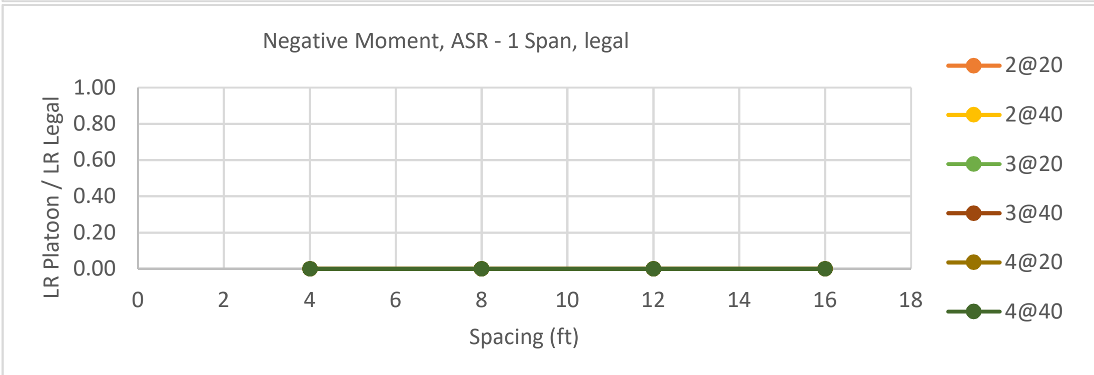


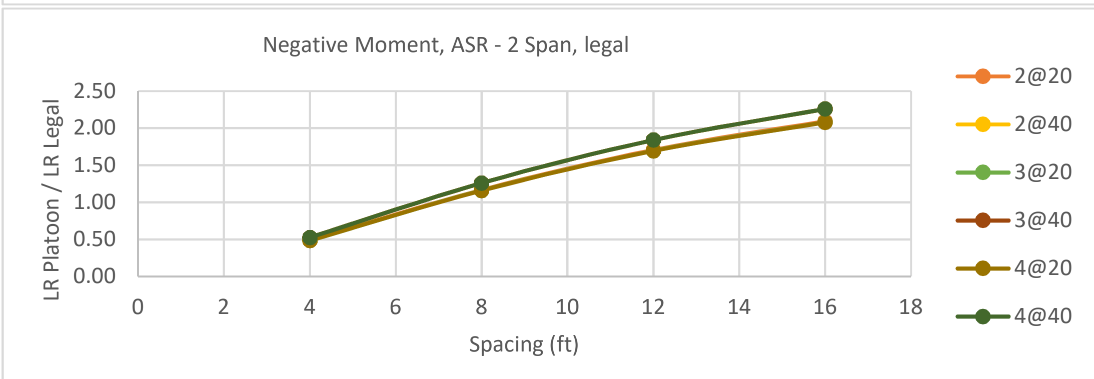
Positive Moment, LRFR - 3 Span, legal
LR Platoon / LR Legal
2.00
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

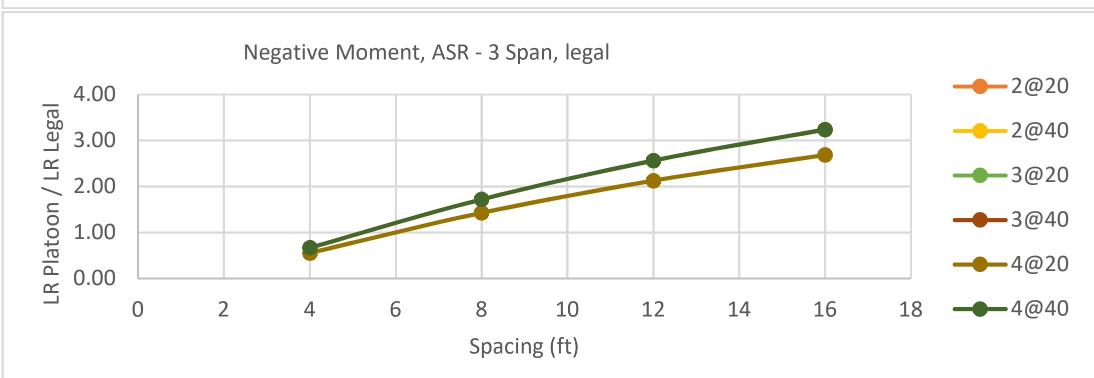


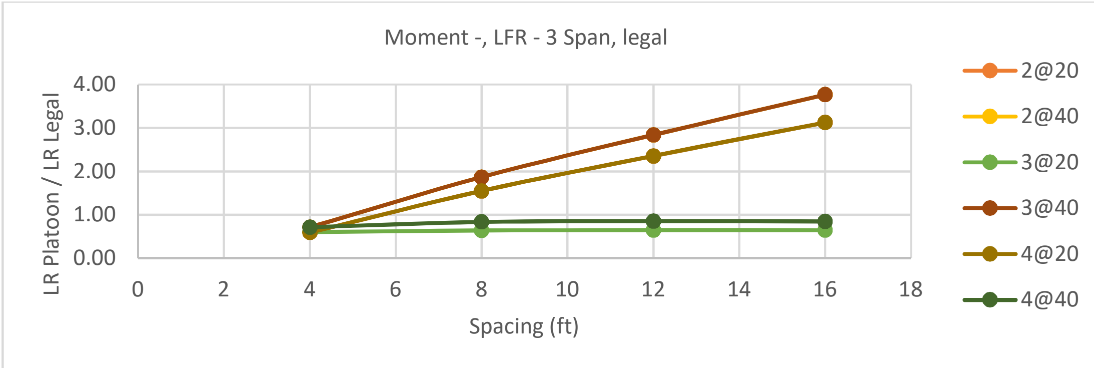
Positive Moment, LRFR - 1 Span, legal
LR Platoon / LR Legal
1.15
1.15
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

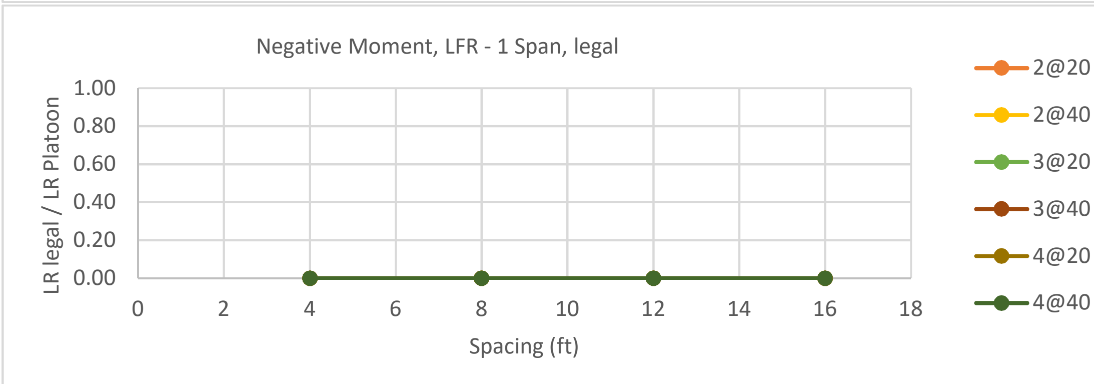

Positive Moment, LRFR - 2 Span, legal
LR Platoon / LR Legal
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

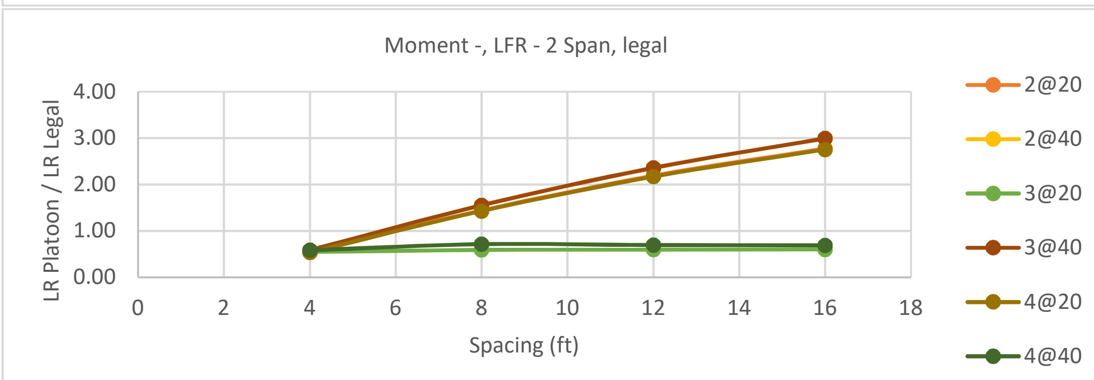

Negative Moment, ASR - 1 Span, legal
LR Platoon / LR Legal
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

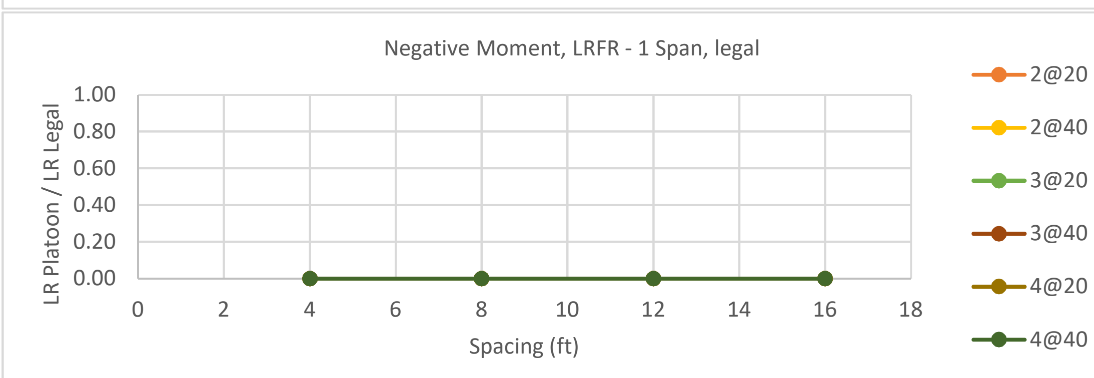

Negative Moment, ASR - 2 Span, legal
LR Platoon / LR Legal
2.50
2.00
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

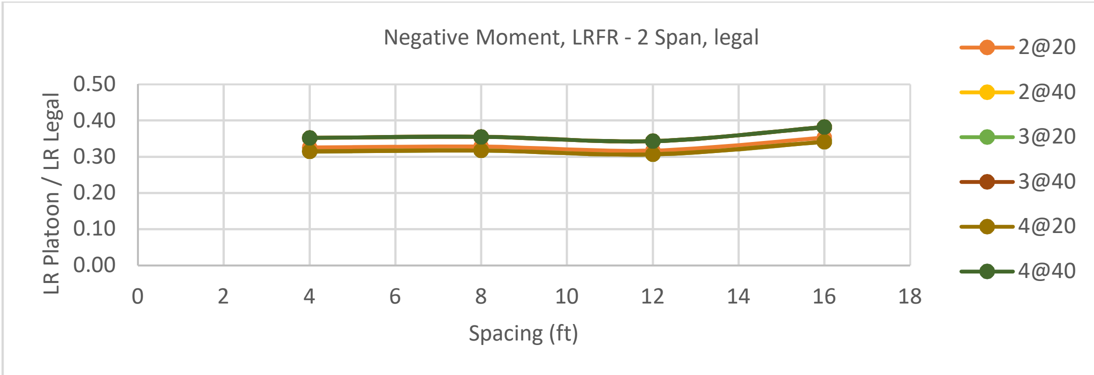

Negative Moment, ASR - 3 Span, legal
LR Platoon / LR Legal
4.00
3.00
2.00
1.00
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

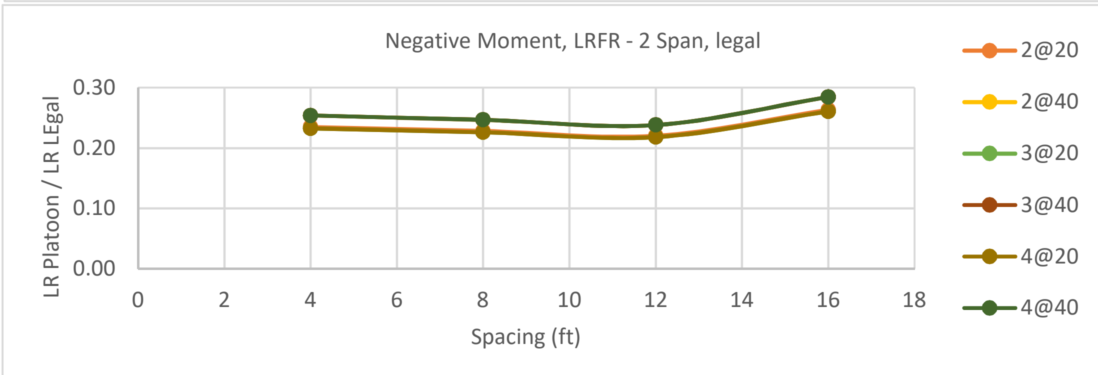

Moment -, LFR - 3 Span, legal
LR Platoon / LR Legal
4.00
3.00
2.00
1.00
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

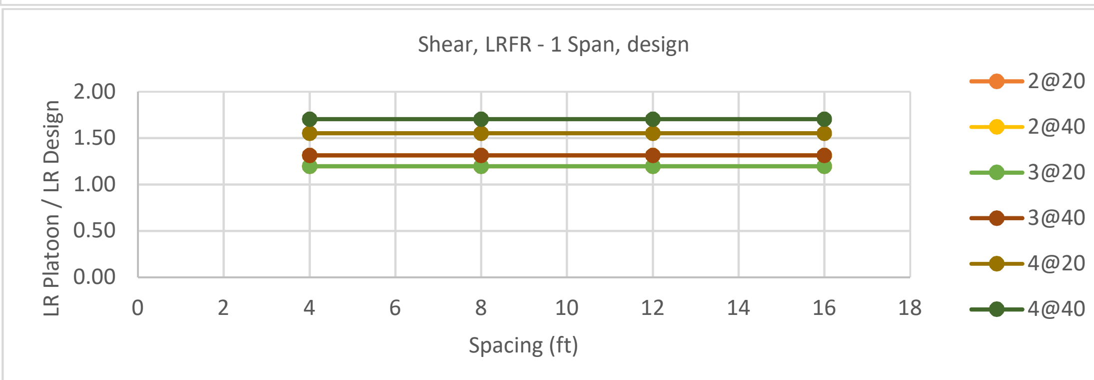

Negative Moment, LFR - 1 Span, legal
LR legal / LR Platoon
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

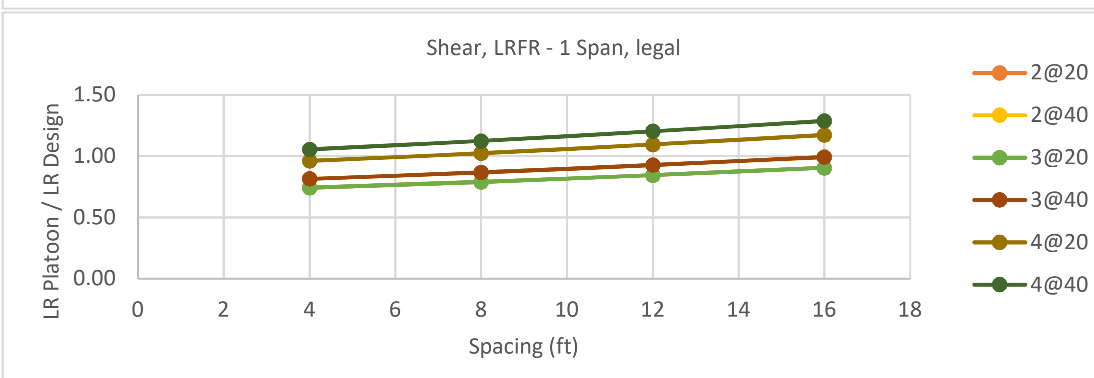

Moment -, LFR - 2 Span, legal
LR Platoon / LR Legal
4.00
3.00
2.00
1.00
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

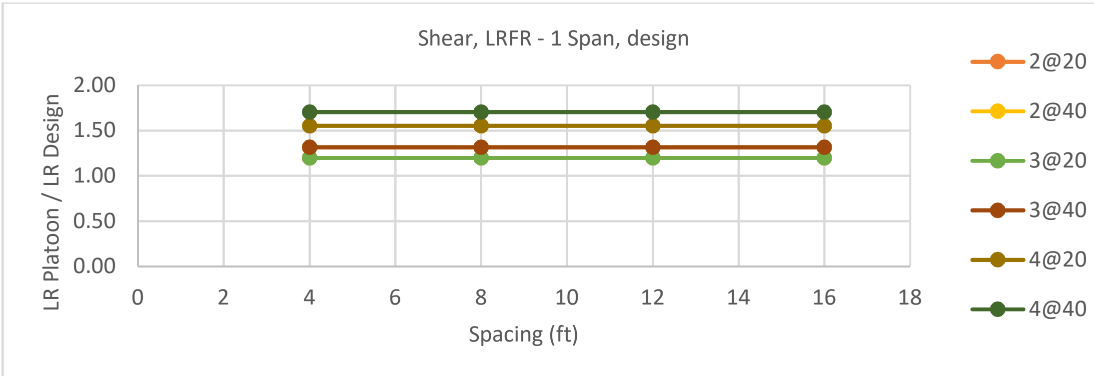

Negative Moment, LRFR - 1 Span, legal
LR Platoon / LR Legal
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

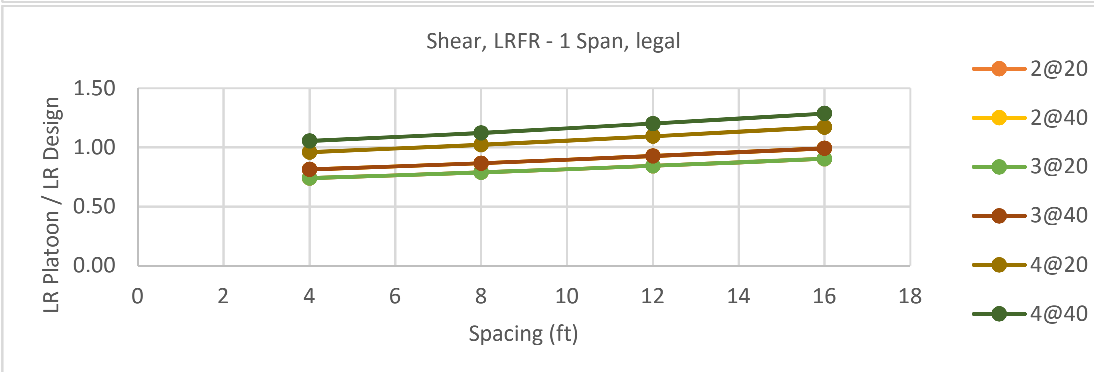

Negative Moment, LRFR - 2 Span, legal
LR Platoon / LR Legal
0.50
0.40
0.30
0.20
0.10
0.00
0 2 4 6 8 10 12 14 16 18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

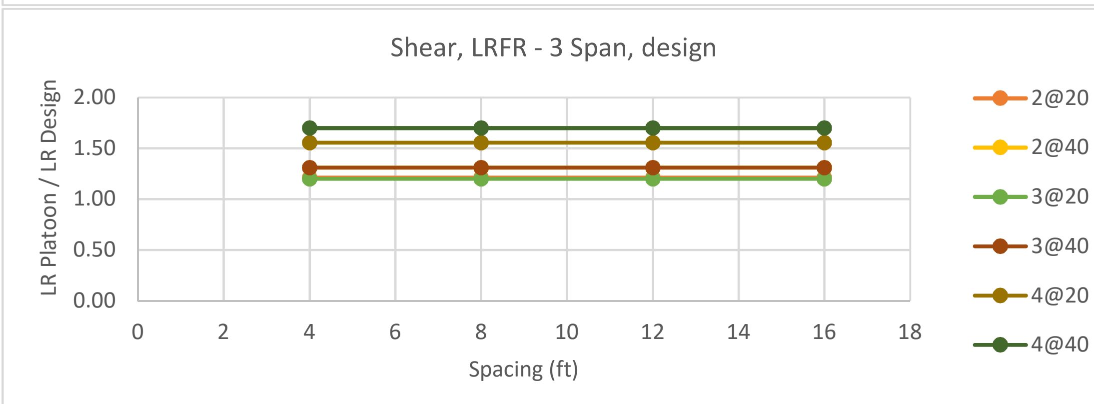

Negative Moment, LRFR - 2 Span, legal
LR Platoon / LR LEgal
0.30
0.20
0.10
0.00
0 2 4 6 8 10 12 14 16 18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40

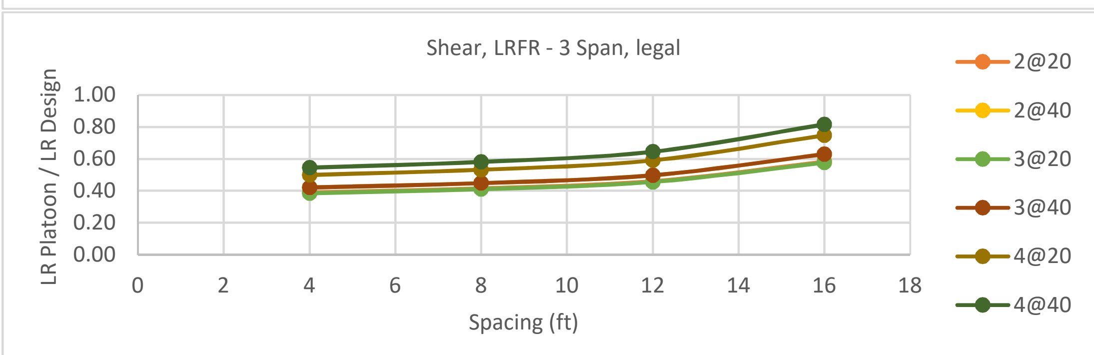

Shear, LRFR - 1 Span, design
LR Platoon / LR Design
2.00
1.50
1.00
0.50
0.00
0 2 4 6 8 10 12 14 16 18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40


Shear, LRFR - 1 Span, legal
LR Platoon / LR Design
1.50
1.00
0.50
0.00
0 2 4 6 8 10 12 14 16 18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40


Shear, LRFR - 1 Span, design
LR Platoon / LR Design
2.00
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40


Shear, LRFR - 1 Span, legal
LR Platoon / LR Design
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40


Shear, LRFR - 3 Span, design
LR Platoon / LR Design
2.00
1.50
1.00
0.50
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40


Shear, LRFR - 3 Span, legal
LR Platoon / LR Design
1.00
0.80
0.60
0.40
0.20
0.00
0
2
4
6
8
10
12
14
16
18
Spacing (ft)
2@20
2@40
3@20
3@40
4@20
4@40